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IRRIGATION PAPERS

OF THE

UNITED STATES GEOLOGICAL SURVEY

No. 40

WASHINGTON
GOVERNMENT PRINTING OFFICE
1900

UNITED STATES GEOLOGICAL SURVEY

CHARLES D. WALCOTT, DIRECTOR

THE AUSTIN DAM

BY

THOMAS U. TAYLOR

WASHINGTON
GOVERNMENT PRINTING OFFICE
1900

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LETTER OF TRANSMITTAL.

DEPARTMENT OF THE INTERIOR,
UNITED STATES GEOLOGICAL SURVEY,
DIVISION OF HYDROGRAPHY,
Washington, September 20, 1900.

SIR: I have the honor to transmit herewith a manuscript prepared by Prof. Thomas U. Taylor, of the University of Texas, upon the Austin dam, for publication in the series of papers upon water supply and irrigation. This relates to a project which in many ways is of great interest to engineers, investors, and citizens having to do with water-power and irrigation projects. There are many useful lessons to be drawn from the history of such an enterprise, for it often happens that failure is more instructive than success. Throughout the United States many communities are now discussing the utilization of water power for irrigation or other industrial purposes, and they may be saved from mistakes or be led to adopt precautionary measures by a clear understanding of the causes of the disasters which have occurred from the neglect of certain precautions.

In this paper the author describes the preliminary projects, the construction of the dam, the difficulties encountered, the silting up of the storage reservoir, and, finally, the failure of the structure and the probable causes which led to the catastrophe. The attempt is made to present these facts from the engineering standpoint and without unduly reflecting upon the motives or characteristics of the individuals concerned. The object is simply to state the facts as they are understood, so that they may be available to persons who are interested in projects of this character.

The necessity of water conservation is so great and public appreciation of the matter has reached such a point that the construction of large dams will undoubtedly be entered upon in various parts of the country, notably in the arid region of the West. There are many places where such structures will without doubt be successful, and there are other cases where the situation must be studied with extreme care and where great caution must be exercised in the preparation of plans and in the selection of locations. The attitude most to be feared is that sometimes adopted by a community where, after discussing the benefits of water conservation, the public as a whole becomes convinced of its importance, and in a condition of excitement

incurs large debts, usually in the form of bonds, to push forward some project not sufficiently matured. The eagerness to attain results leads the people to disregard or pass lightly over the warnings of the engineers or their requests to be given sufficient time to thoroughly work up all of the preliminary information. The public applauds the sentiment of building the structure first and making the plans afterwards, forgetting that the structure is to last an indefinite time, and that a slight error at the outset, due to lack of knowledge, may subject the community to enormous and useless expenditures of money or result in loss of property and life. This is not an imaginary condition, for at the present time several cities or counties of the West are in the position in which the city of Austin was at the time of the adoption of the water-power project. They are urging immediate construction in order to reap the benefit during this time of drought, and are inclined to treat with disdain any intimation that their knowledge of the water supply, of the amount of silt brought down by the stream, and of the character of the foundation of the dam is too vague to justify the incurrence of an enormous debt.

It is for these reasons that this paper is offered for publication. It will serve to answer many of the inquiries made from various parts of the country and to emphasize the importance and practical application of the work of this division of the United States Geological Survey.

Very respectfully,

F. H. NEWELL,
Hydrographer in Charge.

Hon. CHARLES D. WALCOTT,
Director United States Geological Survey.

A. VIEW OF DAM LOOKING EAST TOWARD POWER HOUSE.

B. VIEW OF DAM LOOKING WEST FROM POWER HOUSE.

THE AUSTIN DAM.

By THOMAS U. TAYLOR.

INTRODUCTION.

Austin, the capital city of the State of Texas, is situated on Colorado River, about 200 miles from its mouth at the Gulf of Mexico. The drainage area of this river above the city is 37,000 square miles.[1] The position of the city of Austin with relation to the watershed is shown in fig. 1. This watershed extends from relatively humid regions in the vicinity of Austin westerly into the subhumid or semiarid Staked Plains. From 1856 to 1881 and from 1885 to 1899, inclusive, a period of forty-one years, the rainfall at Austin averaged 32.52 inches, as is shown by the table on page 32. The rainfall over the entire watershed may be assumed to be about two-thirds of this, or approximately 20 inches annually.

A water power was created on Colorado River a short distance above the city of Austin, as related in the following pages. After the dam was completed the project was found to be only partially successful, as the amount of water in Colorado River fell far short of the original predictions. There was at the inception a lack of hydrographic knowledge, especially in regard to the minimum flow of the river, and other information, now known to be vital to the proper location of the dam, was not obtained. Finally, during the great flood of April 7, 1900, the dam was destroyed, with great loss of life and property.

From measurements taken in March, 1890, it was concluded that the minimum flow of Colorado River was 1,000 cubic feet per second. Upon this basis, and upon the assumption that this minimum flow would be held back nights and Sundays and utilized only sixty hours a week, it was concluded that the flow over the Austin dam would develop more than 14,000 horsepower. The city's demands were placed at 2,000 horsepower, and it was the intention to sell to manufacturers the surplus of 12,000 horsepower.

On May 28, 1896, when power was being furnished for pumping, for city lighting, and for city motors, the level of Lake McDonald

[1] The Colorado River of Texas should not be confused with the Colorado River of the West, which is formed by the junction of the Grand and Green rivers in eastern Utah, flows southerly through gigantic canyons, and finally empties into the Gulf of California.

sank below the crest of the dam and remained below until July 6, reaching its minimum (5.7 feet below the crest) on July 1. It was also below the crest from August 6 to August 25 and from September 10 to September 22. This condition was sufficient evidence that the minimum flow could not furnish even 900 horsepower, while 5,227 horsepower had been counted upon. It was apparent that much of the inflow was lost by evaporation, the area exposed to evaporation being 3 square miles and the low level occurring during the hottest part of the summer.

Notwithstanding this evidence that the river could not, during certain seasons, furnish power for its existing load of water, lights, and motors, early in 1897 the Rapid Transit Street Railway and the Dam and Suburban Railway were added to the list of power consumers. After that the energy developed by the water power was

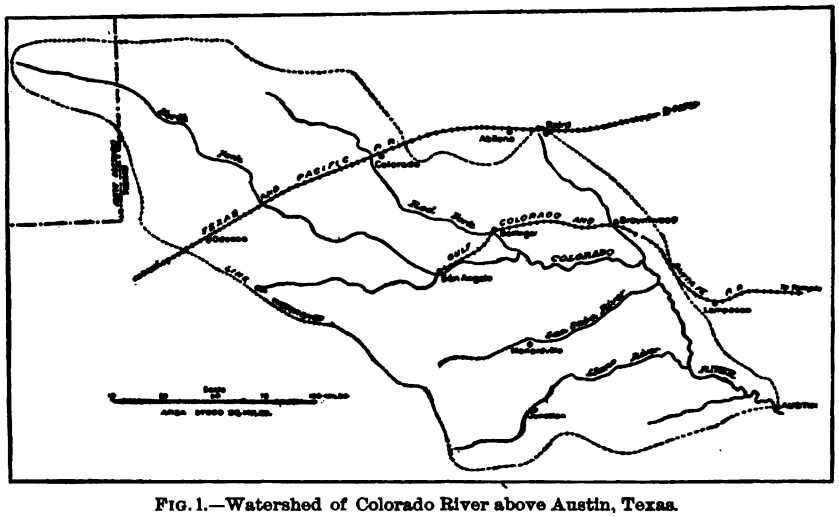


FIG. 1.—Watershed of Colorado River above Austin, Texas.

utilized (1) in pumping water, (2) in furnishing light to city and citizens, (3) in running the Rapid Transit Street Railway, (4) in running the Dam and Suburban Railway, and (5) in running the motors of various users of power in the city—such as for planers, printing presses, etc. To supply all of these demands required a total average of 1,000 horsepower.

On July 4, 1897, the water again dropped below the crest of the dam, but it flowed over again on July 17. It was also below a few days in August, October, and November, and the whole of December. On January 1, 1898, the water was 2.3 feet below the crest, and it did not again flow over until April 13. The lake level was below the crest part of the months of August and September, and it again dropped below on October 10 and continued below until April 20, 1899, reaching a depth of 10.68 feet below crest on March 29. From March 15 to April 17 it was more than 10 feet below the crest. It again dropped

below the crest on August 9, and remained below until October 29, reaching a maximum of 10.45 feet below on October 3. Thus during the year 1899 the water was below the crest of the dam one hundred and ninety-one days, during seventy of which the lake level was more than 10 feet below the crest, and during one hundred and sixty-seven it was more than 5 feet below. The evidence was abundant and unmistakable that the flow was not sufficient to carry the city water supply, the street lighting, the street-car systems, and the motors used in the city. Early in 1899 measurements were made at the head of the lake, at Marble Falls, at the forebay, at the tailrace, and at the station below the railroad bridge. These measurements showed conclusively that the minimum flow was less than 200 second-feet. One second-foot, with an assumed effective head of 57.5 feet and a machinery efficiency of 80 per cent, would develop 5.227 horsepower, showing that an average minimum flow of 192 second-feet would be required to produce 1,000 horsepower, not taking into account evaporation and the leaks through the head-gate masonry ("spring"), which would increase the amount. Again, when the lake level fell more water would be required to develop the same power. When the level was 10 feet below the top, 1 second-foot would develop only 4.3 horsepower; and it is highly probable that the efficiency of the machinery was not as high as 80 per cent.

The result was that the enterprise proved disappointing to a large proportion of the citizens. It is true that it had long been before the people, in an indefinite way, but there was not sufficient data at hand to make the results certain. The watershed and rainfall were rather accurately ascertained; but the keystone of the whole project, the biggest and controlling factor, the very life blood of the system, namely, the minimum flow of the river, was overestimated. The feasibility of the enterprise had been demonstrated more than ten years before, and during the interval accurate and reliable data could have been obtained and the minimum flow ascertained with accuracy. Gage heights, rating tables, and flow curves could have been obtained for one-tenth of one per cent of the outlay.

It was found that the minimum flow could be relied upon to furnish water and lights for the citizens and very little more. It was of great value to the city, from a sanitary point of view, that there was during all these years a private water company, whose plant was operated by steam, supplying water and lights to the citizens.

The history of this dam is unique in one respect, and that is in the number of engineers connected with it. Early in 1892 Mr. Joseph P. Frizell resigned, it is asserted, by reason of the fact that he was hampered in his work by the city authorities. Other engineers resigned for similar causes, and at one time a contractor in charge was ordered to follow the instructions of a city official who was not an engineer. This peculiar method of conducting a great public work called forth severe criticisms from engineering journals.

The failure of the dam to meet expectations and its failure structurally were due to—

(1) The lack of hydrographic knowledge, causing (*a*) an overestimate of the minimum flow, and (*b*) an underestimate of the effect of evaporation.

(2) The hampering of the engineers of construction.

(3) The ignoring of geologic formations.

PRELIMINARY PROJECTS.

Texas became an independent Republic in 1836, and a few years later a commission was appointed to consider the question of locating the capital. In their report they referred to the possibility of developing the water power of Colorado River at Austin.

Nothing was done, however, until in 1871, when Mayor Glenn had surveys made by the city engineer. It was the plan at that time to convey the waters of the Colorado, by a canal, from a point near Mount Bonnel to Shoal Creek, for city and manufacturing purposes.

In 1873 a charter was granted to certain parties, some of whom are still living, to erect a dam across the Colorado, but it was allowed to lapse by limitation.

The drought of 1877 called the attention of the public to the possibilities of irrigating the lands on Colorado River below Austin by means of a dam erected near the city.

During Governor Roberts's administration (1879–1883) estimates were made, with a view to lighting the public buildings, of the cost of erecting a dam across Colorado River at Bull Creek.

In 1888 the board of trade had surveys made for the purpose of determining the feasibility of damming the river, and during the year 1889 the subject was kept before the people by frequent communications in the newspapers.

In the latter part of 1889, the contest for mayor was largely fought on the issue of building a dam across Colorado River. The result of the election foreshadowed early action in regard to the enterprise. In February, 1890, the city council employed Mr. J. P. Frizell to make a report upon the proposed dam and the necessary adjuncts. The report was submitted on March 26, 1890, and so completely does it discuss the whole problem that the portion which refers to the project in general and the part which deals more particularly with the dam, the water power, and the estimated total cost, are here quoted, as follows:

The city is, at present, supplied by a water company, upon what is termed the Holly system; that is, without the use of a reservoir, the pressure in the pipes being maintained by the action of the pumps, which are operated by steam, and increased by an automatic device upon the occurrence of fires. The company also furnishes power for the electric-light system of the city. I have not been able to obtain any very complete information in regard to the extent and size of the present system of pipes. From what I can learn I judge that the city is rapidly outgrowing the capacity of the pipes. That by reason of their small size, a great and increasing burden is laid upon the pumps to maintain pressure sufficient for

A. FRESHET FLOWING THROUGH GAP IN PARTIALLY COMPLETED DAM.

B. RIVER FLOWING THROUGH MAIN GAP IN PARTIALLY COMPLETED DAM.

domestic service in remote parts of the system. A large part of the power of the pumps is consumed in the friction of the water in passing through the pipes: and as the city extends and increases, the protection against fire is becoming more and more precarious. There is also a very widespread impression prevailing that the amount paid for water rates, fire service, and electric lighting is sufficient to supply the city on a much more ample and liberal scale, and at the same time secure incidental advantages of great value.

The project before the city is:

First. The construction of a massive dam across the Colorado to furnish power for pumping, for electric lighting and propelling street cars.

Second. The construction of a reservoir at a height sufficient to maintain fire pressure in the pipes.

Third. The extension of the distributing system on a scale of magnitude commensurate with the present and prospective wants of the city.

Fourth. As an incident of the project it is expected that there will remain a large surplus of power susceptible of such uses as will greatly promote the future prosperity of the city.

THE DAM.

The Colorado above Austin flows in a deep cut or canyon worn in the limestone rock. It is skirted by limestone bluffs rising often to the height of 150 feet above the bed of the river, broken by the erosion of tributary streams. No extensive meadow or bottom lands exist. This situation permits the construction of a high dam with but little damage to private property. The river, in it normal condition, occupies but a small part of the channel in the rock, the remainder being occupied by alluvial deposits to the depth of average high water. In great floods the river spreads from bluff to bluff.

Several situations have been examined with reference to the construction of the dam. One on Taylor's lime chute, about 3¾ miles from the city limits, appears most favorable to the construction of the dam itself, but one on the Brackenridge property, about three-fourths of a mile nearer town, possesses greater advantages as regards the canal and works appertinent to the water power. This site has been selected for the purpose of the estimate.

The channel in the rock is here about 1,150 feet wide at a height of 60 feet above the summer level of the river. The cross section of the channel is not far from level on the bottom, and is bounded by nearly perpendicular walls of rock rising to the height of a little over 60 feet on the city side of the river and 125 or more on the other side. The river bed proper occupies not more than one-half of this width, the remainder of this being alluvial deposit, rising to the height of 40 or 50 feet above the river bed. The situation here is admirably well situated to the development of water power by a dam about 60 feet in height, the perpendicular face of rock rising to about that height, and thence receding from the river in a gentle slope, forming a bench on which the canal or feeder could be constructed, the alluvial strip of ground between the canal and river furnishing sites for pumping and power stations and any other establishments requiring power. An estimate has accordingly been prepared on the basis of a 60-foot dam.

Its crest will be about 1,150 feet in length [the crest was really 1,091 feet long]. It is contemplated to make it some 16 feet thick at the top, increasing downward and spreading out in a broad toe or apron, to give the water a horizontal direction, making its extreme width at the bottom about 50 feet. The body and upstream face of the dam to be made of limestone rock abounding in the vicinity, the upstream face being of quarry-faced work with close joints. The downstream face and toe are intended to be of granite found in abundance in Burnet County, split to approximately regular shape and laid with but a small amount of tooling. The capping is of granite in as large blocks as can be handled, worked to regular shape. The entire work to be laid in hydraulic cement.

The Colorado at Austin drains some 40,000 square miles, and, of course, carries at times an enormous flow of water. The highest flood within the memory of the people now living was some 45 feet above low water, and from the best data I can obtain the flow of the stream was some 250,000 cubic feet per second. This would imply a depth of 16 feet on the crest of the dam, and the abutments should of course go to that height. At one end of the dam the natural rock goes far above that height. The other end is occupied by an artificial bulkhead, called the gate-house, containing the sluices for drawing off the water. It is expected that the wash of the dam during floods will carry away the alluvial deposit for a considerable distance. The wheels, for this reason, must be some 200 or 300 yards from the dam, and the canal must have that length. As already stated, the formation permits this canal to be excavated in rock. At the entrance to the canal is the gate house alluded to above. Its function is to enable the water to be shut out of the canal in case of repairs and to prevent the canal from being overflowed in time of floods. The water will be drawn from the canal through iron pipes, pass the wheel, fall into the wheel pits and be discharged through underground races into the river.

WATER POWER.

It remains to consider the quantity of water power created by the proposed improvement. This consists of two elements, the fall, and the quantity of water available for power. The former is fixed approximately by the height of the dam. The latter can be inferred with more or less certainty from known facts. It is not the lowest stage that the river is ever known to attain to. It is the flow of water that can be depended on, with reasonable certainty, during ordinary seasons. Stages of the river above this minimum count for nothing unless steam is used to make up deficiencies.

The river is subject to great rises in times of heavy rains. On the cessation of the rains it falls rapidly until it attains a minimum flow, which appears to remain nearly constant. In that condition no water enters the stream from the surface of the ground. Its flow is wholly maintained by springs issuing from cavities in the rock, and is unaffected by current rainfall until the latter becomes sufficient to cause a flow from the ground. This is the present condition, and I conclude we shall not be far wrong in taking the present flow of the stream as the quantity that can be depended upon. This, as I have ascertained by careful measurement, is nearly 1,000 cubic feet per second.

There will, no doubt, be times during the hottest weather when the water will fall below this stage, on account of increased evaporation. I am told, however, that a month very rarely passes without rains in some part of the drainage basin, sufficient to cause a slight rise at Austin. The great extent of the pond will enable a considerable deficiency in the flow of the stream to be made good by storage. From the best information I can obtain, the pond will extend some 30 to 35 miles from the dam, with an average width of one-quarter of a mile, containing a water surface of some 8 square miles, and a total volume of something like 2,800,000,000 cubic feet of water. Should the flow of the stream diminish to one-half the above quantity, a single foot in depth on the pond will make good the deficiency for a period of five days, and 6 feet will make it good for thirty days.

A system of flashboards could readily be applied to hold the water 4 feet above the crest of the dam, and thus hold the surplus of water in store for such deficiencies, without drawing the pond below the crest of the dam. This feature will not become necessary for several years, and need not be considered further at present.

Owing to the imperfection of mechanism we can not hope to utilize, for practical purposes, more than 80 per cent of the absolute power of the water. Moreover, the full head of 60 feet can not be brought to act upon the wheels. Some part of the head will be consumed in the movement of the water through the sluices, canal, penstocks, and races. The head will at times be reduced by high water in the

A. CLOSING GAP IN DAM.

B. DAM NEARING COMPLETION, FEBRUARY 1893.

river, which rises more below the dam than above. I therefore take 57½ feet as the head acting on the wheels, and assume that we can utilize 80 per cent of the power on that head. This gives, for the total amount of power available for driving machinery, $\frac{57.5\times62.5\times1,000}{550}\times0.80=5,227$ horsepower.

This quantity of power could be furnished constantly, night and day. This, however, would not be suited to the requirements of industry, which ordinarily calls for power only during the working hours of the secular day. It is regarded as a great advantage in water power to be able to hold back the low-water flow of the stream during nights and Sundays and use it during the working hours. This the great extent of our pond readily enables us to do. Concentrating the entire weekly flow of the stream into the working hours assumed at 60 per week, the above amount is increased in the ratio of 60 to 168, giving as the available power 14,636 horsepower.

The total permanent power of the Merrimac River at Lowell, Massachusetts, is not over 11,000 horsepower, on an average during working hours. About the same at Lawrence, Massachusetts, and at Manchester, New Hampshire. This is the power that can be supplied without interruption. At these points the use of water is not confined to the minimum flow of the stream. It is utilized at much higher stages, in connection with steam, the latter being called into use when the flow diminishes. Of course similar methods will prevail here as soon as the demand for power warrants their introduction. This, however, is too remote for present consideration.

It is not easy to state the rental received for water power at the great manufacturing centers in New England, as grants of water are usually covered with grants of land, the water being regarded as an easement of the land. A round sum was paid for the land and a nominal rent for the water, which was intended as the fund for the maintenance of the appliances of the water power. When manufacturers draw in excess of their grant they are charged all the way from $3 to $12 a day for mill power for water terminable at will. My opinion is that $1,200 per annum fairly represents the value of a mill power. These considerations are adverted to as showing the great value of the incidental benefits secured to the city by this improvement. The city's requirements for pumping I put at 600 horsepower twelve hours per day, or 720 horsepower ten hours a day. Reserving an equal quantity for electric lighting, and equally liberal provision for street cars and other purposes, there would remain over 12,000 horsepower, or as much as 180 mill powers, subject to such use as the city might deem conducive to its prosperity.

ESTIMATED COST.

Dam	$468,325
Gate house	22,950
Canal	49,750
Pump and power house	8,500
Wheel pit	2,123
Culvert	10,210
Wheels and pumps	16,025
Filtering gallery	17,880
Reservoir	104,615
Mains	153,800
Distribution	295,250
Electric light	45,600
Add for contingencies	118,502
Add for engineering and agencies	59,251
Grand total	$1,362,781

On April 3, 1890, the board of public works was created by the city council, to have control of the construction of the dam and of all works connected therewith.

The question of issuing water and light bonds to the amount of $1,400,000 was submitted to the voters of the city of Austin on May 5, 1890, and resulted in 1,354 votes in favor of and 50 votes against the issue. The board of public works met the next day and employed Mr. Frizell as chief engineer and Mr. J. F. Pope as first assistant engineer. In order to guard against possible errors, and to have the advantage of the skill and experience of other experts, Mr. John Bogart, of New York, was employed as consulting engineer, to examine the site and the plans and specifications of Mr. Frizell. After an

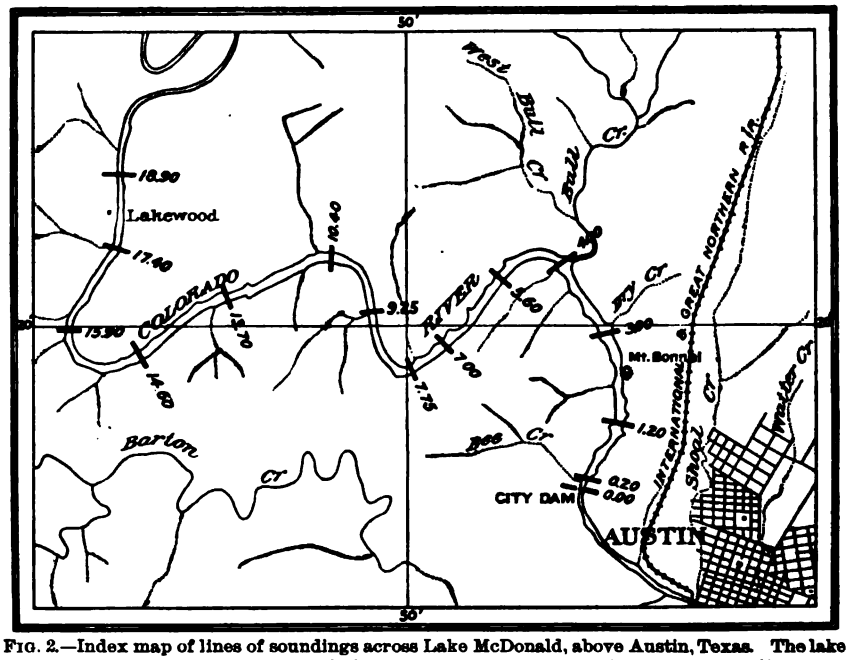


FIG. 2.—Index map of lines of soundings across Lake McDonald, above Austin, Texas. The lake is the broadened portion of Colorado River, extending from sounding 0.00 to sounding 18.90, created by the building of the dam.

extended examination Mr. Bogart made a written report to the board in which he stated that after considerable study he was of the opinion that the site selected by Mr. Frizell was the best place for the location of the dam. In regard to the cross section and shape of the dam Mr. Bogart said:

Ample precedents exist for the determination of the best lines of profile and methods of construction for such a dam. It will be one of the high dams, but will not be among those very high structures which sometimes excite doubt as to their permanent stability. To provide for the possible height of spring or other floods, the whole crest of the dam will be formed so that the water may flow over it, and the downstream face will be built in such a manner and to such lines and curves as will tend to conduct the overflow to the lower river without damage to the stability of the structure. It is estimated that the highest flood as to which any

A. HEAD-GATE MASONRY AND PENSTOCKS BEFORE MAY, 1893.

B. LAYING THE LAST STONE, MAY 2, 1893.

information can be obtained might give a depth of water of 15 or 16 feet over the crest for a short time. Such a possible flow will be provided for in the construction of the dam. The exact lines of the profile and the detail method of construction are now being determined in consultation with Mr. Frizell.

During the summer of 1890 engineers were put in the field to locate the lake-level contours and to determine the amount of valuable land submerged. These engineers took cross sections of the river at sixteen stations, thus enabling comparisons to be made, which was done by the writer in May, 1897, and in January, 1900. The accuracy of the work under the supervision of Mr. Pope was verified by the fact that at the head of the lake, at the request of a farmer living near, he cut a bench mark, on a big pecan tree, at the crest level. Three years later the water, when it rose to the crest of the dam, reached the notch with exactness. Cross sections of the lake are shown in fig. 6; a general map of the lake is shown in fig. 2.

CONSTRUCTION OF DAM.

On October 15, 1890, the contract for the construction of the dam was awarded to the lowest of seven bidders, Mr. Bernard Corrigan, of Kansas City, Missouri, whose figures were $501,150. The contract was based upon the following specifications:

ROCK EXCAVATION.

12. On the site of the dam all unsound rock and all rock that can be removed without blasting will be taken off. A trench 4 feet wide and as deep as may be directed by the engineer will be excavated along the upstream face of the dam. Trenches, footings, steps, channels, and other excavations will be cut in the bottom and sides of the rock in such forms and to such lines as may be directed by the engineer. In these excavations the kind of explosive used, the amount of the charges, depth and direction of the holes, and the entire process of the work shall be under the immediate personal control of the engineer or his assistant, the object being to do the work in such a manner as to avoid fissures and shakes in the remaining rock. All cracks and fissures that may exist naturally or from any cause shall be thoroughly filled with cement mortar or concrete or pure cement, as may in each case be directed by the engineer.

13. The lines and grades of the canal will be established by the engineer, and no excavation below the grade or bottom or outside the lines will be paid for; but such excavation will be permitted, under the direction of the engineer, for the purpose of obtaining rock for the dam.

14. Any rock obtained from these excavations that the engineer may deem suitable may be used in the rubble masonry of the dam. Rock not suited for this purpose will be disposed of as the engineer may direct—in spoil bank, riprap, filling of cribs, or otherwise—not involving a haul of more than 500 yards. The price of rock excavation will include and cover the cost of all the explosives, tools, derricks, tackle, machinery, teams, vehicles, tramways, stringers, bridges, boats and appliances, materials, and labor used in the excavation of the work. All rock will be measured in excavation.

MASONRY.

15. The upstream face of the dam will be laid of granite.[1] It must be sound,

[1] The original specifications called for fossiliferous limestone for the upstream face, but bids were also received for granite, which material was adopted. The details here given are obviously not proper for granite.

free from seams, cracks, and shakes. The stones will be laid on their quarry beds in regular courses, no course to be less than 12 inches in height. Each course shall be composed of headers and stretchers, and at least one-third of the face length of each course shall be headers. The headers and stretchers shall be regularly distributed in each course, so that the headers shall alternate in position in the adjacent courses and so that each stone shall break bond not less than 12 inches with the stone above and below. A gamut or plan of each course will be prepared and submitted to the engineer before the stones are laid and, if approved by him, will be used for the guidance of workmen in laying the stones. This will be quarry-faced work. Each stone will have a distinct pitch line all around the face. The bed and build joints will be fine pointed to a fair, true surface, out of wind, 9 inches back from the pitch line. The remainder of the bed and build joints to be pointed, so as not to project beyond the plane of the fair, true surface and not to fall away therefrom more than 1½ inches. The width of the stretchers on their beds will not be less than their height. No stretcher will be less than 3 feet in length. The headers will not be less than 30 inches on the face nor less than 4 feet in length. This work will be laid with full mortar joints on the beds. The mortar will be pressed or tamped into the build joints with a proper tool. After the work is laid the mortar will be raked out of all the exposed joints to a depth of 1½ inches, and these joints will be pointed with mortar of neat cement pressed in and rubbed hard.

16. In measuring this work, the headers will be accounted 4 feet long. The width of each stretcher will be considered equal to its height. No stone of less dimension will be laid. If stones are laid exceeding these dimensions the excess will be paid for as rubble filling.

17. The downstream face of the dam is to be laid with granite of good quality, sound and free from imperfections. It will be laid in regular courses; no course to be less than 12 nor over 30 inches high. Each course shall be composed of headers and stretchers, and at least one-third of the face length of each course shall be headers. The headers and stretchers to be regularly distributed in each course, so that the headers shall alternate in position in the adjacent courses, and so that no stone shall break bond less then 12 inches with the stone above and below. The stretchers to be as wide as high, measured on the lower bed. No stretcher to be less than 3 feet; no header to be less than 3 feet on the face and 4½ feet long. On the curved part of the face, except at the toe, the bed joints shall be radial. On the vertical and battered part they will be horizontal. Dowels and clamps of wrought iron will be inserted at the toe as directed by the engineer.

18. The exposed faces of all these stones are to have distinct pitch lines all around. The face not to project or recede more than 1½ inches from that line. The beds and builds to be pointed off to lay a three-fourth inch joint for 12-inch back from the pitched line.

19.[1] If this work is laid with fossiliferous marble, the specifications will be the same in every respect as for granite, except that the beds and builds will be dressed to a ⅝-inch joint instead of a ¾-inch joint. Bids will be received for either kind of stone. A gamut or plan of each course will be prepared as specified for the upstream face.

20. In measuring this work the headers will be taken as 4½ feet long, the stretchers as wide as high, measured on the lower bed. No dimensions less than these will be accepted. If stones of greater dimension are laid, the excess will be paid for as rubble filling. All of this work is to be laid in cement mortar.

RUBBLE MASONRY.

21. Rubble masonry will be laid in the body of the dam. This will be composed of any firm, strong, and sufficiently heavy stone. It must not weigh less than 150

[1] This paragraph is eliminated from specifications.

pounds per cubic foot. At least two-thirds of it must consist of blocks having a bed of not less than 4 square feet. Smaller stones may be used to fill up the spaces between blocks. All stones must be bedded in mortar and all interstices filled with mortar. Such practices as laying down stone and pouring grout into the cavities will not be allowed, and the inspector will require such work to be torn up and relaid whenever discovered. The rubble work will be well bonded with masonry on the up and down-stream faces. The trench along the upstream face of the dam will be filled with fragments of granite ["or fossiliferous marble," erased in specifications], of any form and size, carefully laid in cement mortar. Should distinct veins or streams of water enter this trench, the water will be allowed to rise through earthenware drain-pipes set in the masonry, or through openings left in the masonry, and discharged above the dam.

22. The capstones will be of granite of the best and soundest quality, free from all imperfections. They will be 3 feet in depth, not less than 3 feet wide, measured lengthwise of the dam, and each cap will consist of not more than three stones. The stones of the adjacent caps will break beyond 24 inches. A distinct pitched line will be cut around the weather face, following the outline of the stone, as shown on the plan. The weather face will not be more than 1 inch out as regards this line. The beds and builds will be cut to a half-inch joint, 12 inches back from this line. The stones will be bedded in mortar; the build joints will be filled with melted sulphur, of quality approved by the engineer. Dowels of wrought iron will be inserted as indicated on the plan, and also bedded in sulphur: the stones will be fastened together on the top with wrought-iron clamps and dowels, as directed by the engineer.

23. The price of masonry will include furnishing the stone, cement, and sulphur and all materials and labor required for the excavation of the work as herein provided.

24. Before commencing the laying of any masonry the rock on which it is to rest must be swept and washed clean with brooms, and the same must be done when new masonry is joined on to old. All stones must be washed before being laid in the work. A tank and other appliances satisfactory to the engineer must be provided for this purpose.

CEMENT.

25. The cement furnished for this work must be from manufacturers of established reputation, and such as has satisfactorily stood the test of time and experience. It will be subjected to such tests as the engineer may require from time to time, and such as is rejected must be immediately removed from the work. Quick-setting cement will not be used. Cement showing on chemical analysis magnesia or lime in proportions sufficient to injure the work will be rejected. American natural cement will, in general, be expected to exhibit a tensile strength of 50 pounds per square inch at the end of thirty days, when mixed with twice its bulk of sand. Portland cement will be expected to show a strength of 150 pounds per square inch at the end of thirty days when mixed with three times its volume of sand. Cement shall be stored in sheds sufficiently water-tight to exclude rain, with floors raised at least 12 inches above the ground, and permitting a free circulation of air. To give opportunity for tests the contractor shall have no less than sixty days' supply of cement on hand at all times. [Only Portland cement was used.]

MORTAR.

26. The mortar is to be prepared from cement of the quality above described and clean, sharp sand, free from loam or other impurities, in the proportion of 1 part cement to 2 of sand, by measure. If made by hand it is to be mixed dry, and

a sufficient quantity of water afterwards added to produce a paste of proper consistency, and thoroughly worked with hoes or other suitable tools. If required, the contractor shall provide a mortar mill of such form as the engineer may prescribe, to be worked by steam or horsepower. The mixing of the mortar will be under the constant supervision of an inspector. Sand offered for the work shall be washed and screened, if the engineer shall so direct, in such manner as he shall prescribe.

40. And the said contractor hereby agrees to receive the following prices as full compensation for furnishing all materials and for labor in executing all work contemplated in this contract, and for all risks of loss incident to the nature of the work, to floods and freshets on the river, or to other action of the elements, to strikes and combinations of workmen, to changes in market values during the progress of the work, and to all other causes:

(*a*) For bailing and draining, including the construction and maintenance of all necessary cofferdams, the furnishing, setting, and operating of steam engines, boilers, and pumps, and all labor and materials required for the removal and exclusion of water from the work during its entire progress, as specified, the sum of $8,000.

(*b*) For all earthwork excavation above the low-water level of the river, including the disposal of the same in spoil banks or in cofferdams, and all clearing and grubbing, the sum of 14 cents per cubic yard.

(*c*) For all earth excavation below the low-water level of the river, including the disposal of the same, the sum of 50 cents per cubic yard.

(*d*) For rock excavation on the site of the dam, including the disposal of the rock in spoil banks, riprap, or otherwise, as may be required, the sum of $1.60 per cubic yard.

(*e*) For rock excavation from the canal and site of gate house, including the disposal of the rock, the sum of 80 cents per cubic yard.

(*f*) For the masonry of the upstream face of the dam, of granite, cut and laid as specified, the sum of $11 per cubic yard.

(*g*) For the masonry of the downstream face of the dam, of granite, the sum of $11.25 per cubic yard.

(*h*) For rubble masonry laid with stone paid for as excavation, $2.50 per cubic yard.

(*i*) For rubble masonry laid with stone not paid for as excavation, the sum of $3.50 per cubic yard.

(*j*) For rubble masonry laid with granite in trenches, the sum of $6 per cubic yard.

(*k*) For the granite capstones dressed and laid as per specifications, the sum of $15 per cubic yard.

(*l*) For drilling bolt holes for wrought-iron clamps and dowels, the sum of 24 cents per linear foot of hole drilled.

(*m*) For furnishing and inserting wrought-iron clamps and dowels as required by the specifications, the sum of 10 cents per pound.

(*n*) For laying masonry of any kind in Portland cement mortar in excess of the price received for the same work laid in mortar of American natural cement, the sum of 50 cents per cubic yard. [There were more than 90,000 yards excess.]

41. And it is further agreed that the engineer shall make approximate monthly estimates of the work done, and the materials delivered, and the payments shall be made of 85 per cent only of the amount of such monthly estimates.

42. And said contractor hereby further agrees that the said board be, and is hereby, authorized to deduct and retain out of any money due the contractor the sum of $500 per day as liquidation damages for each and every day the aforesaid

work shall remain uncompleted beyond the time herein stipulated for its completion; provided, however, that the board shall have the right to extend the said time, should it decide to do so.

43. And it is further agreed that of the sum agreed on as compensation for bailing and draining, three-fourths shall be considered as earned when the entire masonry is raised to a height of 3 feet above the low-water level of the river, and the remainder when the dam is fully completed according to the terms of this contract, and that these sums shall be subject to the above-mentioned deduction of 15 per cent.

43a. The power of the board of public works and the engineer acting under them as now constituted only extends to executing the contract as it is made by the city; and all changes of the contract and final approval of work done shall be by the city council upon the recommendation of the board of public works, or by the proper city authorities that may be provided by law from time to time as the work progresses.

43b. This contract is conditioned upon a railroad being constructed by the city or some persons other than the contractor from the depots in the city of Austin to the site of the proposed dam, and the said city guarantees to the said contractor the use of the said railroad track free of charge for hauling all material for the construction of the dam. It is understood that all time which may be lost by the contractor by reason of the noncompletion of said railroad shall be credited by said contractor on the time stipulated for the completion of the dam.

The cross section originally adopted is shown in fig. 3, left half. The contractor commenced the work of excavating the alluvial soil on the east bank on November 5, 1890, and the first stone was laid in the foundation of the dam May 5, 1891, exactly one year after the election authorizing the issue of the bonds. The east half of the dam was built to a safe height above ordinary freshets, and then the work was pushed from the west end. The foundation work at the east end was protected by a natural dirt cofferdam and at the west end by a wing wall of broken limestone for body and hay and dirt as fillers. This deflected the water toward the west end of the eastern portion, and the western section of the dam was thus built out to within a short distance of the eastern portion. The gap was closed by first constructing a small dam in front of it, thus forcing the water through a gap in the eastern portion only a few feet higher than the foundation courses. In this way the water was played from one gap to the other at the pleasure of the contractor. The gaps left in the dam while the work of construction was proceeding are shown in Pls. II and III. When the courses reached a sufficient height, a framework dam was made to check the flow through the gaps. In September, 1892, three sluice pipes, each 3 feet in diameter, were built into the dam at a level 43½ feet below its crest, so that there could be no basis for complaint of stopping the flow of the river, and, incidentally, to assist the contractor in controlling the water. At a later stage of the construction one of these pipes, half open and under a head of 36.5 feet, passed the whole flow of the river.

The dam cost over $110,000 more than Mr. Corrigan's original bid. The causes of this increase were—

1. Increased excavation at east end of dam	$8,914.60
2. Extra masonry	56,793.32
3. Extra limestone masonry	21,839.80
4. Extra allowance for Portland cement	44,180.25
5. Change in shape of crest	2,337.45
6. Allowance for pumping, excavating without explosives, and cement	3,385.49
Total	137,450.91
Overestimate	27,255.62
Total extra cost	110,195.29
Original bid	501,150.00
Total cost of dam	611,345.29

The plans recommended by Mr. Frizell contemplated a canal several hundred feet long and a power house south of the site subsequently selected, located upon the alluvial soil below the dam. In January, 1892, it became manifest that some members of the board of public works did not approve the plans submitted by Mr. Frizell in so far as they related to the canal. They advocated the abolition of the canal, and suggested that the water be taken directly from the lake by the penstocks, and that the power house be located near the east end of the dam. At this juncture the board requested Mr. E. C. Geyelin, of Philadelphia, to visit Austin and report upon the questions at issue. In his report, dated February 17, 1892, Mr. Geyelin recommended, among other things, that the power house be located upon a rock foundation at least 100 feet from the dam, and that the water be conveyed from the lake by a series of large pipes. (The power house was afterwards located at an average distance of 130 feet from the dam.) Mr. Frizell took issue with these recommendations, and the board then decided to ask the advice of Mr. J. T. Fanning, of Minneapolis. After spending several weeks in Austin investigating the problems, Mr. Fanning, on June 24, 1892, submitted his report to the board, and among other things said:

This dam is being constructed of solid masonry and is faced on each side with large blocks of excellent granite. Not for its length alone or its great area of flowage is the dam remarkable, for in France we observe three longer masonry dams—at Bouzey, Chazilla, and Gros Bois, 1,545, 1,759, and 1,805 feet long, respectively—and in Wales the Vyrnwy dam, 1,350 feet long, the latter being for the storage reservoir of the Liverpool water supply.

Not in the height alone, for in France there are 3 dams, in Spain 2, in Belgium 1, and in the State of California 1 masonry dam exceeding 150 feet in height. There are 14 other notable masonry dams having heights exceeding 100 feet.

But none of these dams are upon great rivers, and very few of them have any water pass over their crest. On the other hand, the Austin dam is in the channel of the Colorado River, where it has 40,000 square miles of watershed, and will have floods of 200,000 to 250,000 cubic feet of water per second to pass from its crest to its toe. Your citizens will appreciate your responsibility when they learn

that no dam in existence has to pass a volume of water, in flood, even approximating this, through so great a height. Limestone and sandstone yield rapidly to the eroding force of falling waters. The evidences of this are abundant in the canyon of the Niagara River below Niagara Falls, in the canyon of the Genesee River below the Genesee Falls, the Mississippi River below St. Anthony Falls, and here of the Colorado River across Travis County, as well as in the channels of numerous streams that flow down each of the Rocky Mountain slopes. Such evidences admonish us that this great flood must not be permitted to have a sheer fall through so great a height and act with a destructive force such as has heretofore created canyons, but it must be made to glide down the slope of the dam and not be permitted to exert the force due to its velocity except at such distance below the dam that the foundations will not be endangered.

The profile [fig. 3, left half] as shown to me seems not to fulfill the required conditions for passing the floods, because of the slightly rounded or nearly angular form at the front of its crest. Another diagram [fig. 3, right half] presented shows an advised modification of the profile of the upper part of the dam, which is better adapted to pass the flood in a gliding sheet down the face of the dam and to deliver it to the lower level without a direct blow, and so that its velocity will

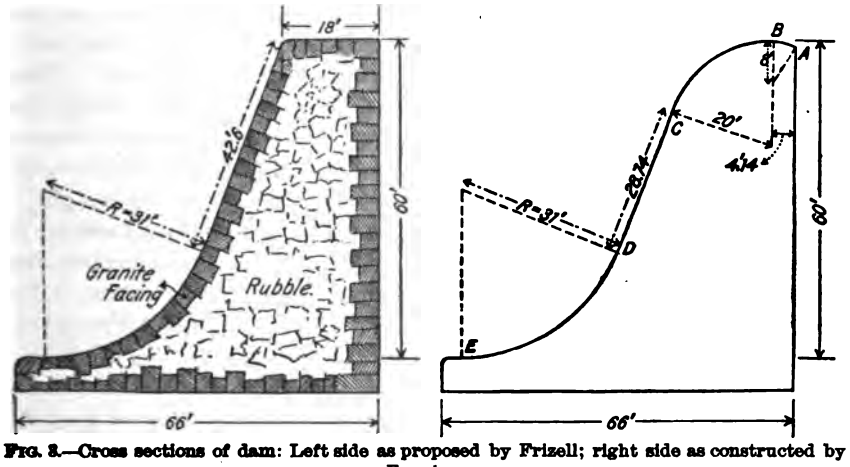


FIG. 3.—Cross sections of dam: Left side as proposed by Frizell; right side as constructed by Fanning.

be expended chiefly in a horizontal direction in the backwater below the dam and in eddies at a safer distance below the toe of the dam. The lower part of the downstream face of the dam has a curve of 31 feet radius to which low-water surface is tangent. The central part of this face has a batter of 4½ inches to the foot.

The new profile at the top part, as suggested, completes the downstream face and crest of the dam with a curve of 20 feet radius, to which both the front batter and the surface of the pond at a level of the crest are both tangent, this curve ending on the crest at 5 feet from the upper angle of the crest. The upper angle of the crest is then rounded off with a smaller curve, and the entire front of the dam becomes a reverse curve of ogee form, the form of dam best of all adapted to pass a large volume of water through so great a height. The top curve conforms nearly to the theoretical form of a medium flood stream. At higher flood stages there will be tendency to vacuum under the curve stream immediately after it has passed the crest, which, together with the pressure of the atmosphere upon the top of the stream, will keep the full flood stream in full contact with the curved face of the dam, and cause even the highest flood to glide down the fall without shock upon the face of the dam or the soft rock foundation.

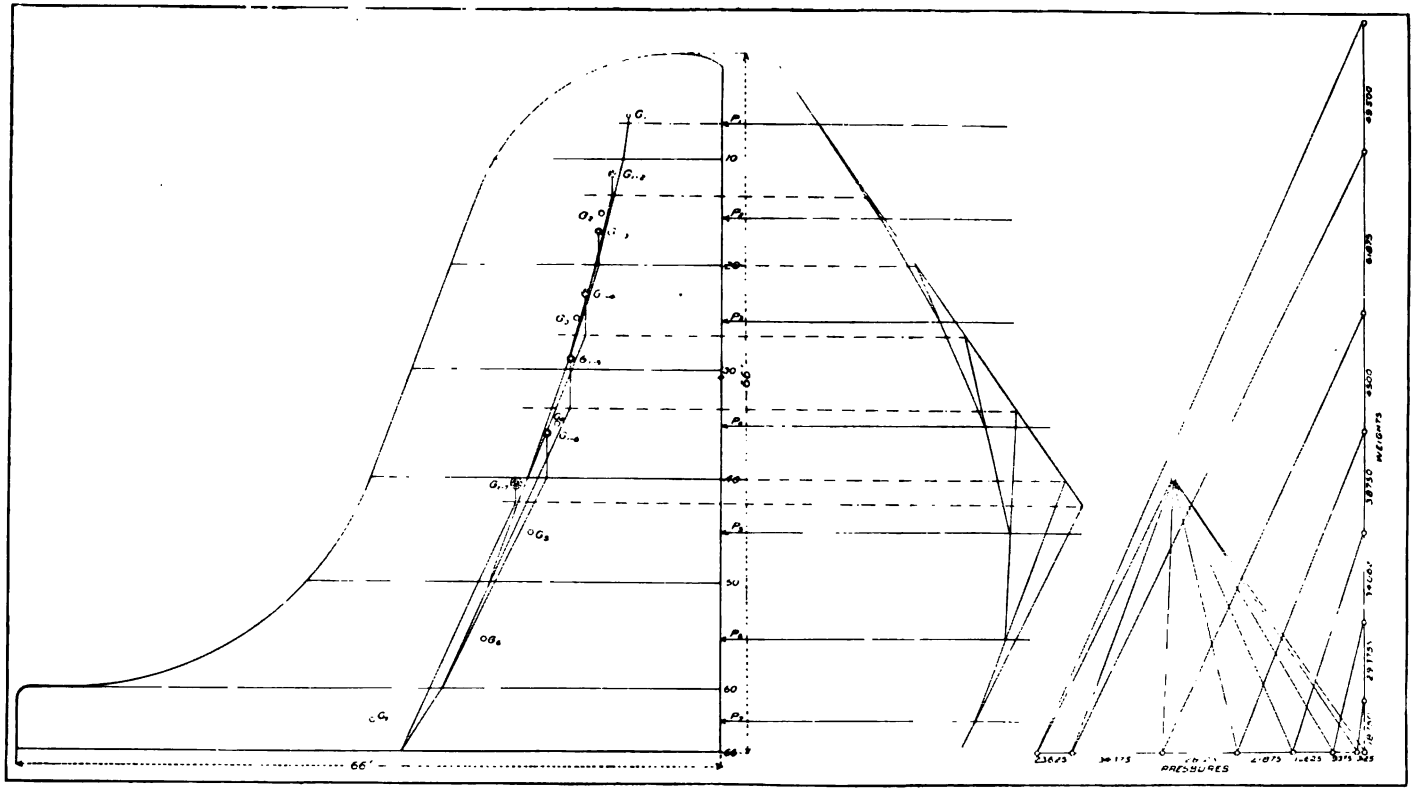
In addition to this, Mr. Fanning recommended that the power house be located in the position shown in fig. 5, and that the river side of the power house rest on a granite revetment wall 250 feet in length, to protect it from the force of the water.

The board of public works finally determined to change the cross section of the dam to that shown in fig. 3, right half, to locate the power house near the east end of the dam, and to take the water from the lake through 9-foot penstocks. The head-gate masonry before May, 1893, is shown in Pl. IV, *A*, which also illustrates the positions of the wrought-iron penstocks.

Pl. V shows the "line of resistance" of the dam, on the assumption that the masonry averages 125 pounds per cubic foot. This line of resistance falls within the middle third, and shows that the cross section was well designed to resist the pressure of the water.

To find the line of resistance of the dam, the height was divided into six equal sections of 10 feet depth, leaving the bottom or seventh section 6 feet deep. The areas of these sections were found to be 150, 235, 271, 310, 360, 495, and 396 square feet, respectively. Their centers of gravity were found and marked G_1, G_2, G_3, etc. For 1 foot in length of the dam the above areas became the volumes of the sections, in cubic feet. The weight of each section was found by multiplying the volume by 125, which was assumed to be the weight, in pounds, of a cubic foot of masonry. While the granite face of the area would have weighed more than this, the limestone cement core would have weighed less. The weights of the sections were 18,750, 29,375, 34,062, 38,750, 45,000, 61,875, and 49,500 pounds, respectively, and they will be referred to in this description as W_1, W_2, W_3, etc.

The horizontal water pressures were found to be 3,125, 9,375, 15,625, 21,875, 28,125, 34,375, and 23,625 pounds, respectively, and they will be referred to as P_1, P_2, P_3, etc. The forces acting upon the first section were the horizontal water pressure of 3,125 pounds (represented by P_1 in Pl. V) and its weight of 18,750 pounds, acting through G_1. By producing the force P_1 and W_1 to intersect, finding their resultant, and producing this resultant to cut the bed joint, we find the first point in the line of resistance. The forces acting on the first two sections were the two water pressures P_1 and P_2 (3,125 and 9,375 pounds, respectively), amounting to 12,500 pounds, and acting as represented by the dot-dash horizontal line between P_1 and P_2; and the sum of the weights (W_1 and W_2) of the first two sections, amounting to 48,125 pounds, acting through their common center of gravity $G_{1\text{-}2}$. By producing these two forces to intersect, and finding where their resultant cuts the second bed joint, we determine a second point in the line of resistance. Similarly, the intersection of the resultant acting on that part of the dam above any bed joint with said bed joint can be found. By joining the consecutive points where the resultants cut their corresponding bed joint the line of resistance is determined.



RESISTANCE LINE OF DAM.

In the case of the Austin dam the line of resistance fell within the middle third of the bed joints, and there was therefore no tension on any bed joint.

By using the force polygon shown on the right-hand side of Pl. V, the work of constructing the line of resistance can be greatly facilitated. Lay off consecutively, in a vertical line, W_1 (the weight of the first section), W_2 (the weight of the second section), W_3 (the weight of the third section), etc., and in a horizontal line lay off P_1 (the first water pressure), P_2, P_3, etc. It is clear that the lines in this diagram joining the ends of W_1 and P_1, W_2 and P_2, W_3 and P_3, etc., are parallel and equal in magnitude to the resultants acting on the first-second, the first-second-third, etc. All that is necessary to find the line of resistance is to produce P_1 to intersect a vertical through G_1, and through the point of intersection draw a line parallel to the first diagonal in the force diagram, to cut the first bed joint, thus determining the first point in the line of resistance. Again, produce the line of horizontal pressure represented by the dot-dash line between P_1 and P_2 to cut a vertical through $G_{1\text{-}2}$, and through the point of intersection draw a line parallel to the second diagonal in the force diagram, to cut the second bed joint, thus determining the second point in the line of resistance. In this way all of the points in the line of resistance can be found.

If it is desired to use graphical methods to find the position of the resultants of the pressures, select any convenient point as a pole and join it to the ends of P_1, P_2, P_3, etc., as shown in the smaller diagram on the right-hand side of Pl. V; then construct the equilibrium polygon, as shown in the diagram to the right of the section of the dam, as follows: First draw a line parallel to the line connecting the pole with the right-hand end of P_1; this line will cut P_1 at any assumed point; then draw a line parallel to the second diagonal from the pole, and passing through the point of intersection of our first diagonal of the equilibrium polygon and P_1; this line will cut P_2; and through this point of intersection a third line of the equilibrium polygon is drawn parallel to the third diagonal from the pole; this diagonal will cut P_3. In this way the complete equilibrium polygon is constructed. The position of the resultants of P_1 and P_2 can be found by producing the third diagonal of the equilibrium polygon until it meets the first diagonal; that of P_1, P_2, and P_3 can be found by producing the fourth diagonal until it meets the first diagonal in the equilibrium polygon; and so on. By drawing lines through these points of intersection parallel to the resultant pressures on the respective sections, we have the positions of the resultant pressures, as shown in the diagram (Pl. V) by the dot-dash horizontal lines. The first pressure in the case before us was not absolutely horizontal, on account of the rounded edge at the top on the upstream face, but it was so nearly horizontal that it will not materially modify the final analysis.

The best modern appliances were used in the construction of the dam. The granite material for the facing was obtained from Granite Mountain, near Marble Falls, being hauled from the quarry to the dam over the Austin and Northwestern Railway, a distance of 70 miles, and delivered at the east end of the dam, as shown in Pl. IV, *A*. The granite blocks were of average dimensions and weighed 4 tons each. The four classes of material used—i. e., the limestone rubble, the cement, the sand, and the granite—were transported from the end of the dam to place by a cable 2½ inches in diameter, stretched between two towers—one on the east and the other on the west bluff—1,350

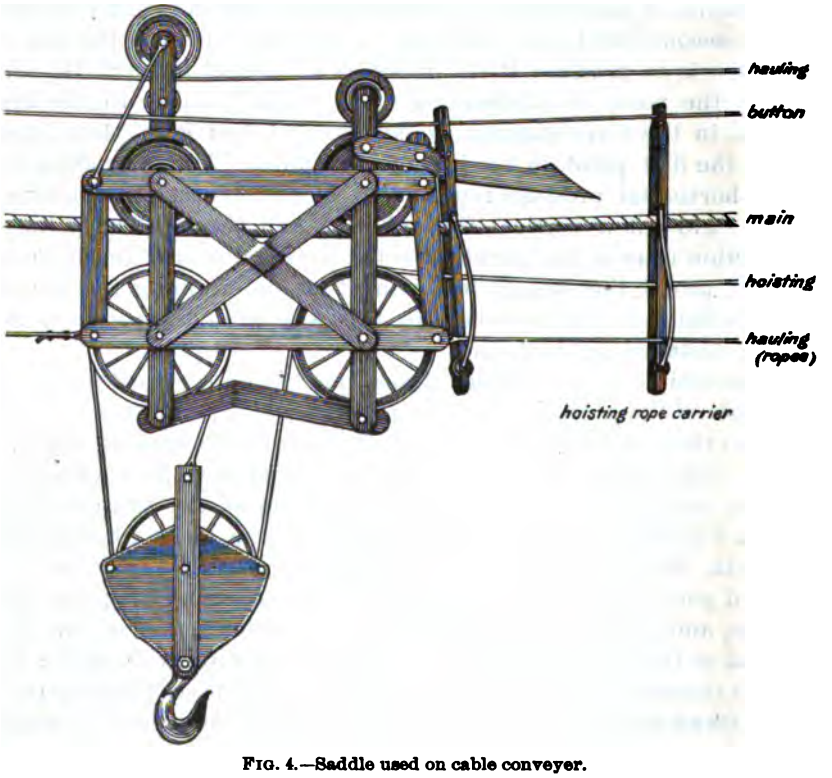


FIG. 4.—Saddle used on cable conveyer.

feet apart. The cable was anchored to "dead men" at the ends, weighted down by stone. The saddle shown in fig. 4 was specially designed for this work, and ran on the main cable. The wire ropes were known as the "hauling rope," the "hoisting rope," and the "button rope." The hauling rope was attached to the lower part of the frame work of the saddle, passed over pulleys at both towers, and wound around a drum under the east tower. The endless hauling rope was operated by an engine to which its drum was attached. It was completely under the control of the operator, and could be stopped in any position along its course. After being checked in the

A. POWER-HOUSE FOUNDATIONS AND OUTLET PIPES FROM TURBINES A FEW DAYS BEFORE FAILURE OF DAM.

B. LIMESTONE STRATA AT HEAD GATE.

position desired, the drum operating the hoisting rope was brought into motion and the load was lowered to the dam.

The granite blocks and the larger limestone rubble stones were handled by immense tong-like grips. The cement and sand were loaded into cages, transported to the place of construction, and there dumped on the dam. The cement mortar was made at the place where it was to be used, and the blocks of masonry were placed where needed by crane derricks shown in Pl. IV, *B*.

A wire rope one-half inch in diameter was used in connection with the cable and saddle to prevent excessive vibration of the operating ropes. On this rope there were buttons which increased in size from the tower to the west. The hoisting rope was supported at different points by carriers which rested, when the saddle was stationary, on the main cable. This carrier consisted essentially of two parallel bars, between which and near the lower end a small pulley was supported to carry the hoisting rope. A series of slots were arranged in the upper part of the carriers through which some of the buttons could pass. When near the east tower the saddle supported all of the carriers on a horn. In moving from the tower to the west, the smaller button passed through all of the carriers except the last, which it took off the horn; the second button passed through all of the remaining slots except that in the second carrier, which it pulled off the horn; etc. The carriers were thus stripped off the horn by the buttons and rested on the main cable, affording a groove or support for the hoisting rope and reducing its vibration.

LEAK UNDER HEAD GATE.

On May 30, 1893, the water from the lake cut under the east bulkhead and undermined the proposed foundation of the power house. It entered a seam in the limestone slightly above the point indicated at B in fig. 5, about 90 feet from the dam. From this point the course of the water was at an angle of about 30 degrees to the axis of the river to the left, and it also took a downward course and passed about 25 feet under the foundation of the head-gate masonry. A view of the power-house foundation a few days before the break is given in Pl. VI, *A*. The water issued from the west wall of the proposed power-house foundation and soon wrecked it. The general course of the water was from a point near the east end of the head-gate masonry diagonally across the foundation to the point of exit.

A cofferdam about 125 feet long was constructed of framework, with dirt and hay as fillers, from a point on the shore above the entrance to the crevice to a point near the end of the dam. This effectually cut off the water from the proposed forebay, and the work of repair commenced. As originally designed, the head-gate masonry contemplated 9 large pipes, but only 7 were put in.

The head-gate masonry cracked about 40 feet from the end of the

dam, along the line HM in fig. 5, and settled. The earth east of the east end of the head gate settled for a distance of 25 feet. Plans were immediately adopted for raising and strengthening the cofferdam so as to provide against floods, for rebuilding the head gate, and for the construction of a power-house foundation. The broken part of the head-gate masonry was removed (leaving only that over penstocks 1

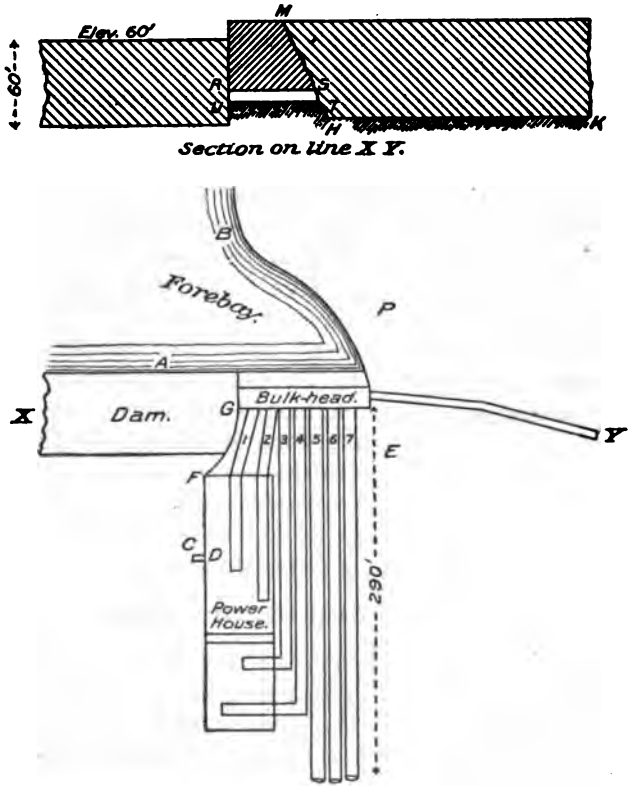


FIG. 5.—Section and sketch plan of bulkhead, showing location of power house and penstocks. HK is bottom layer of concrete filling resting on hard limestone strata; HM is line of crack: RS is the original level of bulkhead masonry; RSTU is tunnel (6 feet by 6 feet by 60 feet long) filled with concrete; A is point where drift indicated leak in the spring of 1899; B is location of leak discovered in the fall of 1899; P is point from which view shown in Pl. VII, *B*, was taken; FG is wing wall; C is 10-inch horizontal pipe projecting from wall of power house, known as "the spring;" D is cement chamber for controlling water and directing it through C; 1 to 7 are penstocks; E is the point at which the leak in penstock was found in February, 1900.

and 2), and an excavation nearly 200 feet long and 70 feet deep, with an average width of 7 feet, was made. This trench reached to a level of 57 feet below the crest of the dam, or within 3 feet of the level of the toe. The head-gate masonry was rebuilt, provision being made for only seven 9-foot penstocks, the rest of the excavation being filled by a concrete wall 112 feet long, which was 8 feet thick for the 90

A. DISCHARGE OF LAKE THROUGH DAM, OR THE SO-CALLED "SPRING."

B. VIEW SHOWING TWO METHODS EMPLOYED TO STOP LEAK THROUGH DAM.

feet next to the head gate and only 5 feet thick for the rest of the distance. In excavating for this extension wall, alternate layers of hard and soft limestone were encountered, as shown in Pl. VI, *B*. The bottom layer of the concrete filling (HK, fig. 5) was laid on one of these hard strata, but it was fully demonstrated that a current of water was running underneath. Several holes were drilled, through which the water welled up in jets several inches high. However, it was thought safe to plug these holes and to ignore the stream below. The bulkhead masonry originally extended to the level RS, fig. 5, 36 feet below the top of the dam; but as an extra precaution a tunnel (RSTU) 6 feet by 6 feet square and 60 feet long was cut under the bulkhead masonry back to the end of the dam proper, and the space was filled with concrete. The space below the 42-foot level under the tunnel was not disturbed.

The foundation for the power house was then excavated to a depth of more than 80 feet below the crest of the dam. The original contractors, after a dispute in regard to the excess of water flooding their work, surrendered their contract. The new contractors succeeded in controlling this water by the use of a cement chamber at D, fig. 5, and a 10-inch horizontal pipe, which projects from the wall of the power house at the point C, about 54 feet below the crest of the dam. This is often referred to as "the spring." A view of it is shown in Pl. VII, *A*, where it is being discharged through a 10-inch pipe in the third course of granite above the toe of the dam. Measurements taken in October, 1895, showed a range, on the horizontal surface of water in the tailrace, of 5.1 feet and a fall of 5.8 feet, giving a discharge of 4.6 feet per second. A 3,000,000-gallon pump entirely exhausted this "spring." In May, 1897, the flow from the so-called "spring" suddenly increased. An average of several measurements gave a range of 11 feet and a fall of 5.8 feet, and therefore a discharge of about 10 second-feet. Measurements recently taken give a range of 8.8 feet and a fall of 5.8 feet, and therefore a discharge of 8 second-feet.

In the spring of 1899 it was discovered, from the behavior of drift, that water was disappearing from the lake at point A, fig. 5. This source of leakage was stopped by filling the lake at that point with clay, loose and in bags. While the filling in was going on the discharge of water from the 10-inch pipe almost ceased for a few hours, but soon reached its normal amount. The filling kept the water discolored and muddy.

In the fall of 1899 it was noticed that water was disappearing at point B, fig. 5, only a few feet from the shore. A cofferdam of sheet piling was constructed around this point and it was filled with hay and earth. A view, taken from point P, fig. 5, of the two leaks and of the method of filling by clay and sheet piling is shown in Pl. VII, *B*. The clay filling is the pile of dirt adjoining the upstream face of the dam.

In the latter part of December, 1899, the earth in front of and between the power house and bulkhead, and at point E, fig. 5, began to sink and continued to sink until it was determined to remove the earth covering the 9-foot penstocks under the sunken areas, in order to discover the cause. The removal of the earth showed that penstock No. 3 (fig. 5) was buckled, and that at one place it had sunk about 20 inches. A bold stream of water—at least 2 second-feet in amount and 16 feet below the top of the dam—was found when the earth was taken out between the power house and penstock No. 3, and between penstocks Nos. 3 and 4. The stream of water was partly deflected around the north end of the power house and under penstocks Nos. 1 and 2. The water did not rise in the excavation, and it was discharged through some 2-inch pipes at F, fig. 5, 14 feet below the crest of the dam, through two or three other small pipes, and by absorption through the wing wall FC and through the east wall of the power house. Penstock No. 5 was also found buckled at a joint about 35 feet from the bulkhead. The water which caused the settling came from a broken 2-inch pipe tapped into the bottom of penstock No. 5. The break was at an elbow just below the penstock, at a point about 16 feet below the crest of the dam and 40 feet from the south side of the bulkhead masonry.

FLOW OF COLORADO RIVER.

The minimum flow of Colorado River was estimated in 1890 to be 1,000 cubic feet per second; and it was upon this estimate that the engineers based their calculations of the power that would become available by the erection of a dam 60 feet high above low water. With an effective head of 57.5 feet and a machinery efficiency of 80 per cent of the theoretic, for this flow there would be available continuously 5,227 horsepower, and if held back nights and Sundays and used only 60 hours a week, 14,600 horsepower.

After February 10, 1897, the city's plant furnished power for its own water and light system, for the Dam and Suburban Railway, for the Rapid Transit Street Railway, and for various motors in the city. This required, in the opinion of the superintendent of water and lights, an average of about 1,000 horsepower; that is, with a machinery efficiency of 75 per cent, about 200 cubic feet of water per second. During 1897 the lake level was below the crest of the dam eighty-nine days, showing that the supply of water at the forebay was not sufficient to carry the load upon it. In 1898 the water was below the crest one hundred and eighty-six days, and in 1899 it was below two hundred and four days, making a total of four hundred and seventy-seven days during the last three years, or 43 per cent of the time from January 1, 1897, to January 1, 1900.

The maximum flood height occurred on June 7, 1899, when the water level of the lake was 9.8 feet above the crest of the dam, giving

A. MARBLE FALLS, ABOVE LAKE McDONALD

B. VIEW UP RIVER FROM MOUNT BONNEL, AFTER DESTRUCTION OF DAM.

a flood discharge of 101,000 cubic feet per second, or enough water passing each second to run the plant eleven days.

From March 15 to April 17, 1899, the lake level was more than 10 feet below the crest of the dam; and again, from September 16 to October 23, it averaged about 10¼ feet below the crest. In all, it was more than 10 feet below the crest of the dam during seventy-two days in 1899, or about one-fifth of the year. During these periods of low lake level the following measurements of the flow were taken:

Discharge measurements of Colorado River during low lake level.

Date.	Locality.	Gage height.	Discharge.
1899.		*Feet.*	*Second-feet.*
January 31	Head of lake	—7.60	210
March 13	Marble Falls	—9.85	197
March 15	Forebay at dam	—10.00	206
Do	Tailrace at dam	—10.00	233
Do	Forebay at dam	—10.05	186
March 17	Tailrace at dam	—10.05	203
Do	Between bridges	—10.05	267
October 3	Head of lake	—10.45	134
October 4	do	—10.35	134
October 6	Marble Falls	—10.25	105
October 4	Head of lake	—10.35	136

In explanation of these measurements it is necessary to say that the low level of the lake compelled the authorities of the city to cut off the power from the Rapid Transit and the Dam and Suburban railways on March 20 and September 9, and from the tower lights on January 22 and September 6. The flow (206 and 186 second-feet) into the penstocks was then being utilized for pumping water, for operating the street-car systems, and for running a few motors in the city. The effective head on the turbines was about 50 feet, giving a theoretic horsepower of 850. The actual horsepower was about the same. The flow below the dam, at the tailrace, was in excess of the inflow by 27 and 16 second-feet, respectively. The flow through "the spring" pipe was 10 second-feet, and the rest can be accounted for by the momentary fluctuations in the supply to the turbines that ran the street-car systems. This fluctuation was caused and controlled by the demand made by the motors and the cars for power. The inflow was controlled by automatic gates that responded to these demands.

The difference between the inflow and the flow at the station between the bridges was 81 second-feet. Barton Springs supplied 20 second-feet of this, the power-house "spring" 10 second-feet, and other small springs along the river between the dam and the railroad bridges supplied a large part of the remainder.

While the city plant was furnishing power for all of the purposes mentioned, the average flow at the tailrace was about 300 second-feet,

which should have developed 1,500 horsepower. During the day the flow was less than during the active hours of the night, but measurements taken on the tailrace gave a flow of 295 second-feet, with the turbines which ran the city's pumps, the street-car systems, and the city motors in full operation.

All measurements at the station between the railway and the highway bridges were so completely controlled by the flow through the penstocks at the dam that they gave no indication of the unrestricted flow of the Colorado.

The minimum flow was not sufficient to carry the load of the pumps, the lights, two street-car systems, and the motors for power plants in the city, and when this flow was decreased by the evaporation from the lake surface (3 square miles) the lake level sank rapidly.

The low stages of the water in 1899 can not be ascribed to a decreased annual rainfall but to a very irregular one. For the purpose of comparison, the rainfall at Austin since 1856, with the exception of the years 1882, 1883, and 1884, records for which are not obtainable, is given below.

Rainfall at Austin, Texas.

Year.	Rainfall.	Year.	Rainfall.	Year.	Rainfall.
	Inches.		*Inches.*		*Inches.*
1856	19.6	1870	42.5	1887	23.5
1857	33.0	1871	30.1	1888	51.8
1858	36.4	1872	33.3	1889	43.2
1859	28.2	1873	43.4	1890	36.4
1860	29.6	1874	46.5	1891	37.1
1861	28.7	1875	29.3	1892	34.7
1862	22.2	1876	32.6	1893	16.3
1863	34.7	1877	41.8	1894	27.5
1864	25.2	1878	21.6	1895	31.9
1865	37.8	1879	18.3	1896	21.9
1866	40.5	1880	48.1	1897	28.1
1867	27.3	1881	25.3	1898	25.9
1868	40.1	1885	40.3	1899	28.0
1869	38.5	1886	32.4		

The rainfall of 1899 (28 inches) was greater than that of 1898, nearly equal to that of 1897, and greater than that of 1896; but the rainfall of that year (1899) was concentrated and not distributed like that of previous years. In 1899 very heavy rains fell in the months of April and June. In the latter month occurred the highest flood within the last twelve years.

In the discussion that arose over the proposition to change the cross section of the dam, as shown in fig. 3, the question of the amount of flow and its effect on the toe of the dam was considered. Mr. Frizell then brought to the attention of the engineering public the following proof of the formula of flow: Let H = the height of lake level above crest of dam; x = the part of this height that gives the velocity over the crest; Q = flow in second-feet; v = velocity in feet per second;

A. VIEW OF DAM FIVE MINUTES BEFORE FAILURE ON APRIL 7, 1900.

B. FLOOD POURING OVER DAM ON JUNE 7, 1899 LAKE LEVEL 9.25 FEET ABOVE CREST OF DAM.

l = length of crest; $H - x$ = depth of water on crest: Then $v = \sqrt{2gx}$ where $2g = 64.32$ feet; $\therefore Q = v\,(H - x)\,l = l\,(H - x)\,\sqrt{2gx}$. This must be a maximum, hence the derivative of Q with respect to x must be zero; hence—

$$O = l\sqrt{2g}\left[\frac{H - x}{2x^{\frac{1}{2}}} - x^{\frac{1}{2}}\right],\ \therefore x = \tfrac{1}{3}\,H,\ \therefore Q = 3.09\,l\,H^{\frac{3}{2}}.$$

In the case of the Austin dam, $l = 1091$, $\therefore Q = 3362\,H^{\frac{3}{2}}$.

The formula generally used to find the flow over crests is $Q = cl\,H^{\frac{3}{2}}$. With a view to testing the reliability of this formula and finding the coefficint c, in January and March, 1900, measurements of velocity were taken, with a small Price electric meter, by Mr. E. E. Howard, a senior in the engineering department of the University of Texas. The dam broke before the investigations were completed, but the following values of the coefficient c were found:

Tabular statement of values of coefficient c.

Date.	H.	h. (a)	v.	c.
1900.				
January 15	1.09	0.92	3.85	3.09
Do	1.09	.92	3.99	3.21
Do	1.09	.92	3.85	3.09
Do	1.09	.79	4.22	3.14
January 18	.72	.625	2.33	3.06
Do	.72	.625	2.33	3.06
Do	.72	.625	2.37	3.11
Do	.72	.625	2.29	3.00
Do	.72	.625	2.33	3.06
Do	.72	.625	2.33	3.06
Do	.72	.626	2.33	3.06
Do	.72	.625	2.29	3.00
Do	.72	.625	2.37	3.11
Do	.72	.625	2.29	3.00
Do	.72	.625	2.33	3.06
January 26	.42	.33	1.57	3.06
Do	.42	.33	1.61	3.13
Do	.42	.33	1.61	3.13
Do	.42	.33	1.61	3.13
March 28	1.44	1.04	4.95	3.32
Do	1.44	1.04	4.95	3.32
Do	1.44	1.04	5.01	3.36
Do	1.32	.96	5.27	3.33
Do	1.32	.96	5.18	3.26
Do	1.32	.96	5.27	3.33
Do	1.32	.96	5.18	3.26
Do	1.32	.96	5.27	3.33

a Depth of water on crest.

Floods of historic importance occurred as follows:

	Feet.
February, 1843	36
March, 1852	36
July, 1869	43
October, 1870	36
June, 1899	23
April, 1900	33

On July 3, 1869, began the longest and most uninterrupted rain ever known in Austin. It rained without stopping for about sixty-four hours, and the river rose to a height of 45 feet. The lower part of the city was overflowed and several people were drowned. The town of Webberville, 16 miles below Austin, was also overflowed, and Bastrop, 30 miles below, was inundated.

ECONOMIC ASPECT.

The population and taxable wealth of the city at various times is shown in the following table:

Population and taxable wealth of Austin, Texas.

Year.	Assessed valuation of real estate.	Assessed valuation of personal property.	Total assessed value.	Increase each year.
1889	$5,665,886	$2,324,757	$7,990,643	
1890	6,462,209	2,473,343	8,935,552	$944,909
1891	7,577,116	2,936,962	10,514,188	1,578,530
1892	8,054,405	2,719,318	10,773,723	259,635
1893	8,184,297	2,697,633	10,881,930	108,207
1894	8,384,429	2,640,342	11,025,368	144,335
1895	8,520,659	2,863,075	11,384,734	259,366

Before the issuance of the $1,400,000 of water and light bonds in 1890, the bonded indebtedness was as follows:

Six per cent bonds due in 1901	$12,500
Six per cent bonds due in 1905	40,000
Six per cent bonds due in 1925	72,500
Total	125,000

On May 5, 1890, the city authorized the issuance of $1,400,000 water and light bonds at 5 per cent. The first $400,000 were sold on October 15, 1890, to a syndicate of local capitalists at par and accrued interest. In April, 1892, the Union Trust Company bought $500,000 of the bonds at 95 cents on the dollar, and in 1893 Bernard Corrigan bought $62,000 of them at 95. Later, in 1893, $388,000 were sold at 92 and $50,000 at par.

Total receipts from bonds.

450 bonds, at par	$450,000
562 bonds, at 95	533,900
388 bonds, at 92	356,960
Total	1,340,860

The break in the head-gate masonry and the destruction of the first foundation of the power house caused an outlay of $97,000 above all expectations, and it was apparent that another issue of bonds would be necessary to complete the enterprise as originally contemplated.

A. VIEW THREE MINUTES AFTER FAILURE OF DAM.

A—B and B—C are portions of dam first broken, H—A is eastern part of dam (83 feet long on crest) left standing

B. VIEW TEN MINUTES AFTER FAILURE OF DAM.

D′—B′ is broken portion of dam still standing; B′—C′ is western portion of break; C is end of western part of dam (456 feet long on crest) left standing.

Under the direction of the board of public works the following expenditures were made:

Disbursements.

Engineering expenses	$58,402.37
Dam	611,313.39
Electric-light dynamos	7,700.55
Electric-power generators	6,708.50
Belting	43.81
Power house	45,917.98
Penstocks, head-gate castings, etc	47,792.19
Draft-tube excavations	2,314.08
Pump foundations	653.66
Bonds	1,237.50
Head gates	2,122.60
Sluice pipes	2,420.19
Repairs on account of break	96,941.23
Wheels, pumps, etc	43,418.02
Countershafting, pulleys, etc	1,203.85
Water distribution system	158,081.04
Electrical distribution system	115,678.29
Railroad	87,431.90
Office expenses	10,170.03
Submerged lands	27,732.15
Head-gate masonries	46,934.17
Power-house foundations	11,527.60
Miscellaneous	5,375.74
Total	1,391,129.64

The board of public works was discontinued on the completion of the dam in 1893, and the city council then assumed control and managed the plant until the water and light commission was created by charter in 1897. At the time the board of public works was abolished there was due and unpaid on contracts the sum of $55,896.87. The water and light commission had exclusive supervision, management, and control of the waterworks system, the electric lights, the power plant, and all property, funds, and business belonging or pertaining thereto.

The $200,000 of bonds issued in 1895 were utilized in paying the $56,000 indebtedness left by the board of public works, and in completing the water and light system, with the exception of the reservoir. The reservoir was never built. After a site had been practically chosen it became evident that it would be necessary to filter the water of the lake, and upon the advice of Mr. Allen Hazen, of New York City, the sanitary engineer called in for consultation, it was determined to construct filtering galleries in the sand flats about 2 miles below the dam, and to transfer the pumps to a new pump house to be erected near the filters. These galleries, three in number, were in successful operation when the dam broke. The lower station was

equipped with a 6,000,000-gallon pump and a 300-kilowatt synchronous motor, which cost $29,380. The filtering galleries and connections cost, in round numbers, $21,000.

No better idea can be given of the operation of the plant than to append a report for the twelve months ending November 30, 1899:

Earnings:	
Water	$40,369.39
Light	30,192.61
Power	12,777.14
Miscellaneous	1,206.86
Total	84,546.00
Collected in cash	67,298.12
Due by city	12,745.19
Garnisheed accounts	2,836.70
Due on bills, etc	1,665.99
Total	84,546.00

For the purpose of comparison, the receipts and expenditures for the last three years are here tabulated:

Receipts and disbursements of power plant at Austin, Texas.

Year ending—	Receipts.	Operating expenses.	Extensions.	Total disbursements.
November 30, 1897	$82,059.44	$36,709.07	$57,821.02	$94,530.09
November 30, 1898	93,651.05	36,239.32	38,724.54	74,963.86
November 30, 1899	82,927.43	39,742.31	34,711.95	74,454.26

SILTING UP OF LAKE McDONALD.

In 1890 cross sections of Colorado River were taken at sixteen stations, as shown in the tabular statement on page 38 and in fig. 6. The complete outline of the diagrams in fig. 6 represents the original cross section, the horizontal line being the water surface even with the crest of the dam and the shaded area showing the amount of silt that had been deposited up to February, 1900. All vertical dimensions are exaggerated three times over the horizontal dimensions. Cross sections were again taken for the United States Geological Survey in May, 1897, also in January, 1900. The silt deposited from 1893 to 1897 is represented by the lower shaded area, and that deposited from 1897 to 1900 by the upper shaded area.

The water first flowed over the crest of the dam on May 16, 1893, at which time there were 83,556,000 cubic yards of water in the main channel of the lake up to the level of the crest of the dam; in 1897 there were only 51,889,000 cubic yards of water in the channel, the remaining 31,667,000 cubic yards (or 38 per cent of the original capac-

A. VIEW ABOUT TWO HOURS AFTER BREAK.

B. VIEW ONE HOUR AFTER BREAK.

ity) being silt. Estimated in depths on a square-mile base we have, in 1893, a volume of water equal to a depth of 80.9 feet, and in 1897,

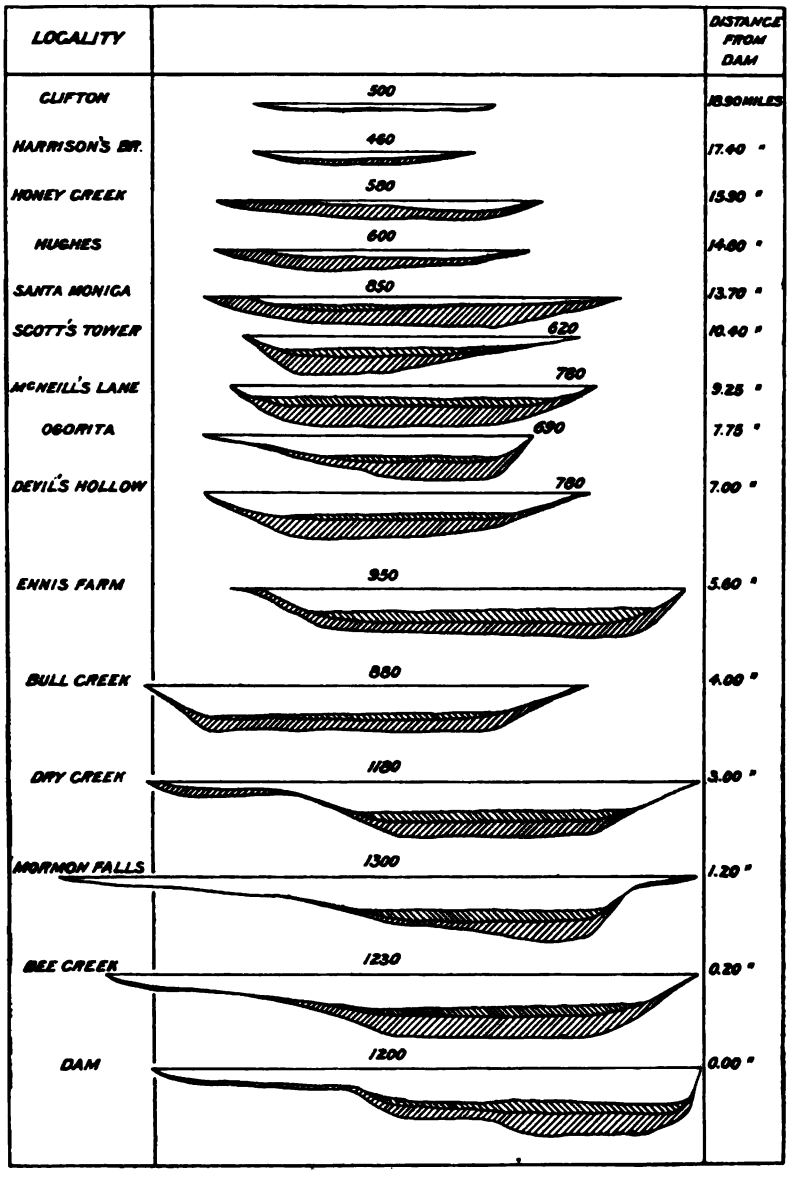


FIG. 6.—Cross sections of Lake McDonald, illustrating accumulation of sediment between 1893 and 1897 (illustrated by the lower shaded area) and between 1897 and 1900 (illustrated by the upper shaded area). The number above each section indicates its length, in feet.

four years later, we have a volume of water equal to a depth of 50 feet and silt to a depth of 30.9 feet, showing the average amount of silt deposited annually to be 7.7 feet, on a square-mile base.

The following table shows the maximum and mean depths of water for 1893 and 1900, the maximum and mean depths of silt for 1900, and the percentage of silting up at the respective stations:

Table showing silting up of Lake McDonald.

Station.	Maximum depth of water.		Maximum depth of silt, 1900.	Mean depth of water.		Mean depth of silt, 1900.	Amount of silting up to February, 1900.
	1893.	1900.		1893.	1900.		
Miles.	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Per cent.*
0.0	66.0	39.0	27.0	40.1	27.5	12.6	31.2
0.2	67.0	38.0	27.7	37.6	26.5	11.1	32.2
1.2	63.8	35.5	28.3	30.6	21.7	8.9	30.2
3.0	56.0	31.5	24.5	33.4	21.5	11.9	38.8
4.0	47.8	31.5	19.0	38.0	23.5	14.5	37.9
5.6	47.5	27.5	23.0	36.7	23.0	13.7	42.0
7.0	47.0	22.0	26.0	36.3	17.4	18.9	56.3
7.75	44.8	20.5	25.0	30.7	15.4	15.3	56.3
9.25	40.4	13.4	29.0	30.8	10.2	20.6	66.9
10.4	40.9	13.5	27.4	27.2	7.9	19.3	71.8
13.7	29.4	9.8	26.0	20.3	5.5	14.8	78.9
14.6	24.0	12.5	16.0	17.2	9.0	8.2	50.4
15.9	16.6	12.0	15.0	13.2	10.0	3.2	60.0
17.4	13.2	9.5	7.0	11.2	8.8	2.4	30.0
18.9	7.6	5.0	2.6	5.6	3.9	1.7	33.0
20.0	3.7	2.2	1.5	2.8	2.0	1.8	35.0

The maximum depth of silt is not always equal to the difference between the maximum depth of water for 1893 and 1900, as the channel shifted at several points. This is very noticeable at station 13.7, known as the Santa Monica (or Sulphur) Springs station. The last column ("Amount of silting up," etc.) gives the ratio that the present cross section of silt bears to the original cross section of water. Thus at Santa Monica Springs (station 13.7) 78.9 per cent of the original cross section has filled with silt.

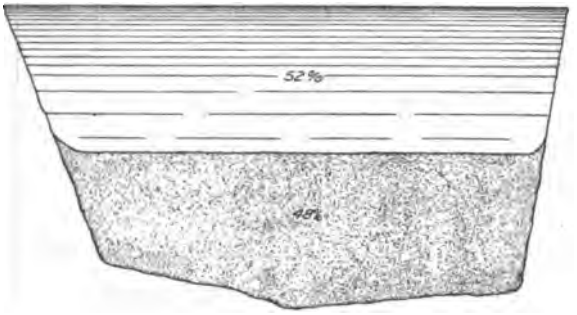


FIG. 7.—Generalized cross section of Lake McDonald, showing accumulation of sediment.

In February, 1900, there were 43,460,000 cubic yards of water in the main channel of the lake beneath the level of the top of the dam (equivalent to 42.1 feet on a square-mile base) and 38.8 feet of silt. Thus 48 per cent of the original storage capacity of the lake was at that time mud. Up to that date the average rate of deposit, on a square-mile base, was 5.8 feet per year.

A. VIEW ABOUT 350 FEET BELOW DAM, SHOWING WAVES THOUGHT TO BE DUE TO SECONDARY DAM FORMED BY DÉBRIS FROM WRECK.

B. POWER HOUSE OF AUSTIN WATER, LIGHT AND POWER COMPANY ABOUT 2 MILES BELOW DAM.

In 1897 this silt, to within 2 miles of the head of the lake, was a fine, impalpable, absolutely gritless deposit, and where newly exposed would not bear an appreciable weight on its surface. The writer has often tried its resistance all along the lake, and an oar could be driven into it several feet with moderate pressure. Shovelfuls of it placed upon boards in a heaped-up mass would immediately settle and spread so that the upper surface was almost horizontal. A barrelful of it, when first taken up at Santa Monica Springs, soon spread out in a flat sheet. At the head and for about 2 miles down the lake the silt consisted of a sand which readily deposited when the velocity of the stream was checked by the waters of the lake. At occasional points below the head of the lake small bars of sand were found near the mouths of small canyons or creeks.

From March 15 to April, 1899, the water level of the lake was a

FIG. 8.—Layer of silt remaining on plateau after destruction of dam.

little more than 10 feet below the crest of the dam. The water again commenced flowing over the crest of the dam on April 21 and continued to flow over, at small depths, until June 7, when the river rose to a height of 9.8 feet above the crest of the dam. This flood continued until June 12, and its effect on the cross sections near the head of the lake was marked. The sections at stations 14.6, 15.9, 17.4, and 18.9 were scoured out 2 to 3 feet deeper than the sections of 1897, and at station 15.9 a sand bar was deposited on the inside (left) of the curve of the river, contracting the channel to less than half its former width.

The typical section illustrating the ratio of silt and water areas for the whole lake is about midway between stations 5.6 and 7.0, i. e., about one-fourth of a mile below the Chautauqua wharf. Fig. 7 illustrates this section, the vertical scale being magnified ten times.

Fig. 2 shows the configuration and geography of the lake formed by

the dam. The river, as shown by fig. 1, for 200 or 300 miles, flows through a hilly country, from above Colorado City, on the Texas and Pacific Railroad, and in its course absorbs the waters of the Concho, the San Saba, the Llano, and the Pecan Bayou. All of the country drained by these tributaries is hilly, with the exception of a few miles along the head of the Colorado and the Concho.

When the break occurred the silt in the immediate vicinity of the dam would have flowed out had there been no water. Just above the part of the dam that gave way was a plateau whose surface was on an average about 18 feet below the crest of the dam. At the time of the break the lake level was 11 feet above the crest of the dam, making the depth of water on the plateau 29 feet. The torrent poured over this plateau with immense velocity, as shown in Pls. X and XI. The silt on the plateau was cut away with such swiftness that in three hours it was swept almost clean. Only a slight amount remained, as can be seen from fig. 8; the men are standing on the original soil of the plateau, the silt appearing behind them.

In the main channel the upper surface of the silt was 38 feet below the crest of the dam, giving, at the time of the break, a depth of water in the main channel of 49 feet. After the water level dropped below the plateau the current was confined to a narrow gorge. The flood continued in the main channel for several days. A week after the failure the silt along the shores of the former lake was cut into fantastic shapes by the currents of the river and those of many mountain gorges. The silt in contact with the dam undoubtedly increased the pressure against it, but that portion of the dam across the main channel where the silt was 28 feet deep and where the pressure was greatest was on a good hard rock foundation and successfully resisted it. There was practically no waterlogged drift in this silt; the soundings indicated mud bottom. The silt deposit kept the river at Austin muddy for months after the failure. (It is still muddy—November 1, 1900.)

Pl. VIII, *B*, is a view of the lake from Mount Bonnel, looking up the river, two weeks after the break, with the freshet still on. The extreme right-hand part of the curve is the mouth of Bull Creek.

For purposes of comparison, the silting up of reservoirs is best reduced to heights and depths on a square-mile base. To derive an expression for the amount of silt deposited in a given time, let x equal depth, in feet, of silt deposited in a year by each foot of water in the reservoir, and let h equal the original depth of the reservoir. The depths of water are at the end of one, two, three, etc. n years:

First year, $h-hx=h(1-x)$;
Second year, $h(1-x)-h(1-x)x=h(1-x)^2$;
Third year, $h(1-x)^2-h(1-x)^2x=h(1-x)^3$;
Fourth year, $h(1-x)^3-h(1-x)^3x=h(1-x)^4$.

Hence, if we let d=depth of water in n years, we have $d=h(1-x)^n$.
In 1897 $h=81$, $d=50$, $n=4$; $\therefore x=0.1135$.
In 1900 we have $d=0.52h$, $n=6\frac{2}{3}$, $(1-x)^{\frac{20}{3}}=0.52$; $\therefore x=0.09343$.

A. VIEW UPSTREAM ONE DAY AFTER FAILURE OF DAM.

B. VIEW WESTERLY ALONG LINE OF DAM ONE DAY AFTER FAILURE.

For safety let us assume the least value of x; the following table gives the results for each year on that assumption:

Table showing silting up of reservoirs.

n years.	Amount of water, $d+h$.	n years.	Amount of water, $d+h$.
1	0.907	12	0.308
2	.822	13	.279
3	.745	14	.253
4	.676	15	.230
5	.612	16	.208
6	.555	17	.189
6½	.520	18	.171
7	.503	19	.155
8	.456	20	.191
9	.414	25	.086
10	.375	30	.053
11	.340	40	.020

These results will not be correct for any reservoir in which there is an appreciable current acting on the bottom of the basin, i. e., on the upper surface of the silt. In Lake McDonald the level of the water could sink to 10 feet below the crest of the dam and there would still be a fair current in the penstocks, as the bottom of the forebay was 12 feet below the crest. When this condition obtained, as it did during the months of March, April, and October, 1899, there was a current on the upper third of the lake. Unless the lake was drained by the three 3-foot pipes at the west end of the dam (which did not occur after 1893), it was not possible for the current under ordinary conditions to affect the silt in the lower two-thirds of the lake. The results of the table are illustrated in fig. 9.

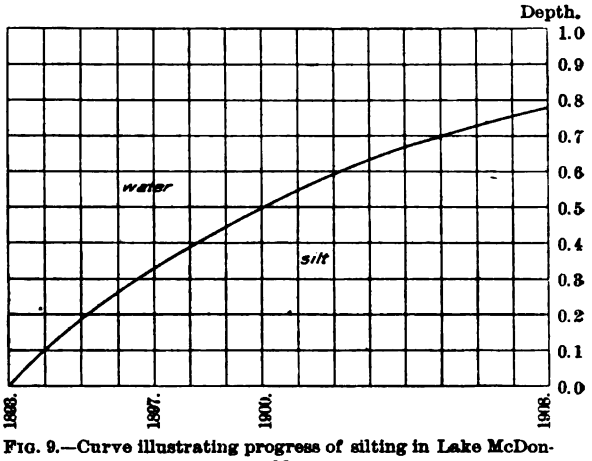


FIG. 9.—Curve illustrating progress of silting in Lake McDonald.

FAILURE OF THE DAM.

At Austin the Colorado emerges from a mountainous country which extends for a distance of over 200 miles in a northwesterly direction. The channel above Austin is for the most part a sinuous gorge held in by limestone mountains and hills. Austin is at the foot of a long range of mountains, 37,000 square miles of which afford a drainage area for Colorado River. The river is fed by the Perdinales, the

Llano, the San Saba, the Concho, and the Pecan Bayou. The configuration of the country is such that the water runs off rapidly into streams.

From 1 p. m. on April 6 to 4 a. m. on April 7 there was a rainfall of 5 inches at and in the vicinity of Austin, in a mountainous country and on ground already wet. In addition to this, tremendous rains fell all along the Colorado and its tributaries from Austin as far up as Llano, a distance of over 100 miles. The river rose rapidly, and by 10 a. m. on Saturday, April 7, it was apparent that the dam would be called upon to withstand the biggest flood since water first wetted its crest on May 16, 1893. At that hour the water level of the lake was more than 10 feet above the crest of the dam and it was rising nearly 2 feet an hour. The greatest previous flood height occurred at 9 p. m. on June 7, 1899, when the lake level was 9.8 feet above the crest of the dam. The water rose rather gradually throughout the day to its maximum level of 9.8 feet. Pl. IX, *B*, gives a view of the flood of June 7, 1899, taken when the lake level was 9.25 feet above the crest of the dam and the water level only 22 inches lower than that of April 7, 1900; and Pl. IX, *A*, is a reproduction of a snapshot picture taken with a small kodak five minutes before the dam broke. It shows conclusively that there was no drift in concentrated masses. Isolated logs and débris were passing, but they never came within 8 feet of the crest of the dam.

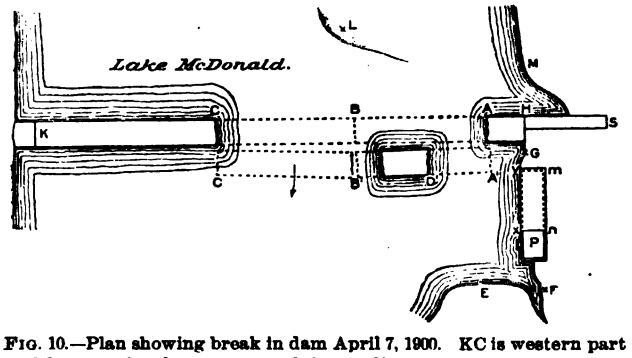


FIG. 10.—Plan showing break in dam April 7, 1900. KC is western part of dam (456 feet long on crest) left standing; HA is eastern part (83 feet long on crest) left standing; AB and BC are portions of dam first broken; xymn is dynamo room at power house; P is pump room; L is point from which view shown in Pl. XV, *A*, was taken; F is point from which view shown in Pl. XIII, *A*, was taken; G is point from which view shown in Pl XIII, *B*, was taken; M is point from which view shown in Pl. XI, *A*, was taken; E is point from which view shown in Pl. IX, *A*, was taken; HS is bulkhead; B'C' and D'A' are portions of dam that broke and disappeared 45 minutes after break; B'D' is the broken portion of dam still standing; S is point from which view shown in Pl. XI, *B*, was taken.

At 11.20 a. m. on April 7, when the lake level had reached a height of 11.07 feet above the crest of the dam, the dam gave way at the point marked B in fig. 10, about 300 feet from the east end of the dam. Observers at E, F, and H all agree in their testimony that it first opened at B, and as though the mad current had simply pushed its way through the structure. Sooner than it takes to write these words the two sections AB and BC, each about 250 feet long, were shoved or pushed into the lower positions A'B' and B'C', about 60 feet from

A. VIEW OF POWER HOUSE ABOUT TWO HOURS AFTER FAILURE OF DAM.

B. WRECK OF POWER HOUSE.

their former positions in the dam. There was not the slightest overturning. After the warning break at B, the water over the part ABC was seen to rise several feet, and the next instant the pent-up waters were pouring over the sections A'B' and C'B'.

The view shown in Pl. X, *A*, was taken at 11.23 a. m., or three minutes after the failure. The parts may be identified by comparing Pl. X, *A*, with fig. 5 and with the sketch plan, fig. 10, the view shown in Pl. X, *A*, extending slightly beyond C in fig. 10. By 11.30 a. m. the lake level had fallen to the crest of the dam, and the sections A'B' and B'C' were seen to be upright, each still a solid mass, complete, unbroken, and intact, except for the scaling off of the granite facing from the downstream side, occupying a position practically parallel to the dam, a view of which is shown in Pl. X, *B*, a snapshot taken ten minutes after the break. To the casual observer the detached portions at this time had the appearance of having been erected in their new positions by the original contractor. The crests of the sections A'B' and B'C' were about on a level with their original positions, except at C, where the crest was slightly higher, giving B'C' a slight longitudinal dip toward the power house. Measuring along the crest the break left 456 feet of the dam (KC in fig. 10), at the west end and 83 feet (AH) at the east end still standing unaffected.

As soon as the sections were broken out and moved to the positions A'B' and B'C', the partially pent-up waters rushed through the gap, those held back by CK producing a strong current in the direction of the power house. This current struck the wall of the power house almost on a level with the floor of the pump room (about 12 feet below the crest of the dam), crushed in all of the windows on the west side, flooded all of the lower stories, and caught and drowned five employees and three small boys. Two of the employees miraculously escaped by climbing through a belt hole in the dynamo room (x y m n, fig. 10). These workmen were pumping water from the lower portions of the power house.

At 12 o'clock, forty minutes after the break, the broken section B'C' turned over toward the dam and disappeared beneath the torrent, and the eastern end of the section A'B' was broken up and engulfed. This left about 100 feet (B'D'), which was shifted slightly out of its parallel position. This section (B'D') was cracked from the crest as far down as could be seen. Sometime during Saturday night the smallest portion of this 100-foot section was swept away. The picture reproduced in Pl. XI, *A*, was taken from the top of the bulkhead at 2 p. m. on Saturday, April 7.

At 12.05 a. m., Sunday, April 8, the northern two-thirds (x y m n, fig. 10) of the west wall of the power house gave way and fell, taking with it the roof over the dynamo room and wrecking the corresponding part of the east wall. The power house was 198 feet by 54 feet, and had a total height on the river side of 112 feet. The walls were

of brick, except the first 20 feet, which were constructed of dressed granite rocks about 2½ feet by 5 feet by 3 feet. The dynamo floor (x y m n, fig. 10) was about on a level with the crest of the dam. The floor of this room extended as a gallery over the pump room (P) in which the big duplex pumps were located.

The broken sections moved back about 60 feet and the toe rested on the shore line of the tailrace. The impact and the resultant compressive stress on the stones on the curved face tore the granite facing loose from the core in several places. The section was stripped of its facing stones in the third course from the top as far down as could be seen, a distance of fully 30 feet. Some of the top granite stones of the section were loosened in their beds by the break and were carried away by the current.

Specimens of the cement of the core have been examined and these have been found to be first class in every particular. In some places breaks occurred through limestone rubble and the adhering cement, showing that the strength of the joint was superior to that of the limestone.

The behavior of the granite facing indicates very clearly that it was easily pulled off by the immense forces brought into play. The impact cracked the section along an irregular surface about 42 feet below the crest.

There was not sufficient continuity between the rubble core and the granite facing. There were slight irregularities in the line of contact between the granite and the rubble, but when subjected to the powerful forces of the water pressure the two separated at many places. In a few cases isolated granite blocks on the curved crest were forced from their beds, as shown by the dark spot on the curved crest in Pl. X, *B*.

The maximum depth of the water near the dam was only 38 feet below the crest. Thus at the time of the break the total depth was a little more than 49 feet. The lake had silted up from the original bed-rock bottom in the main stream exactly 30 feet. Had there been no silt a much larger volume of water would have passed and the results have been more disastrous at and below the dam. While this silt would flow, it was sluggish and served to retard the current, thus prolonging the flood several hours (it actually continued, with great velocity, for nearly two days). Within twenty-four hours the river level had fallen more than 40 feet, but this only served to confine the flood to the main channel, with little diminution of velocity. At 3.20 p. m., four hours after the dam broke, the water level had dropped 30 feet and was at the high-water mark of the old channel, cutting the banks both above and below the dam. Below the dam the alluvial banks reached a height of 64 feet above the toe of the dam. This whole mass was in a few hours cut back 40 feet. The effect of this continuous flood on the silt deposited in the lake since 1893 is considered under the heading "Silting up of Lake McDonald," page 36.

A. WESTERN PORTION OF DAM REMAINING AFTER BREAK.

B. PORTIONS OF BROKEN DAM BROUGHT DOWN BY FLOOD.

It is almost certain that the dam failed by sliding. It seems that at a point 300 to 400 feet from the east end the limestone upon which the dam rested was of a friable nature. Mr. Frizell realized this, and it was stated to be a part of his plan, had he continued in charge of the work, to reenforce the bottom of the river just below the dam by a cement foundation about 100 feet wide by 600 feet long; it was also contrary to his purpose to have the water of the tailrace run along the toe of the dam.

In order to ascertain whether any of the foundation of the dam remains in the broken gap, soundings were made in June and in the latter part of October, 1900. Three lines (A, B, and C) of soundings were made parallel to the upper face of the dam, namely, at 2, 16.5, and 42 feet, respectively, from the upper face. A cord was tagged every 20 feet, and the soundings were made with a sharpened three-fourths-inch iron rod. Stations were taken west to east. The following table shows the result of the soundings, the depth being measured from the crest of dam. When the October soundings were made the water surface was 56.1 feet below the crest of the dam. The rod soundings were added to this to give the depth recorded in the table.

Result of soundings taken at Austin dam site in June and October, 1900.

Station.	Line A.		Line B.		Line C.	
	Depth in feet.	Bottom.	Depth in feet.	Bottom.	Depth in feet.	Bottom.
60					Water's edge.	Dam.
40					63.1	Solid rock.
20					67.9	Do.
0	Water's edge.	Dam	Water's edge.	Dam	68.4	Mud.
20	73.5	Solid rock.	69.1	Solid rock.	69.5	Solid rock.
40	69.5	do	69.1	do	70.1	Do.
60	69.9	do	70.1	do	70.9	Do.
80	70.6	do	71.1	do	71.3	Do.
100	71.1	do	72.6	do	72.5	Do.
120	72.3	do	72.6	do	72.4	Do.
140	a 72.6	do	72.9	do	72.6	Do.
160	a 72.3	do	73.1	do	73.5	Do.
180	a 71.9	do	72.1	do	73.1	Do.
200	a 71.3	do	71.1	do	71.3	Sand.
220	a 70.9	do	69.1	do	71.0	Rock.
240	a 68.1	do	68.9	do	69.6	Mud.
260	Deep sand.		67.1	Mud	68.1	Gravel.
280	do		63.6	Gravel	67.9	Mud.
300	Water's edge.		63.1	do		Loose rock.
320						Water's edge.

a Sand was found, but the sounding rod was driven through to solid rock.

If we remember that the height of the dam above the rocky bed of the river was 66 feet, and that the foundation was not (except as noted on the next page by Mr. E. W. Groves), even with its toe-hold, more than 68 feet below the crest, a glance at the foregoing table will convince anyone that no part of the foundation in the western 300 feet of the broken section remains. From the present ordinary eastern water's edge a large sand bank extends along the entire eastern section still remaining. Soundings were not made through this 30-foot sand bank, but the crest of the big section of the dam, shown in all of the views of the broken part, is only 4 feet lower than the crest of the portion standing, which indicates very clearly that its foundation went with it, as it is resting practically in the old tailrace, whose bed was lower than the bed of the dam.

On April 8, 1896 (four years before the dam failed), in a letter to the mayor, Mr. Frizell called attention to the fact that dangerous abrasion might occur near the point mentioned, i. e., 300 to 400 feet from the east end. The waters of the tailrace followed the toe of the dam fully 600 feet and were discharged through a narrow neck 45 feet wide, at an average depth of 2.8 feet during the day and about 3 feet at night when the full power was on. In March, 1899, the writer, with Mr. H. K. Seltzer, made soundings along the toe of the dam between this narrow neck and the power house. At or near the point where the break occurred the bottom was not reached with a rod 5.3 feet long, even when the hand holding it was thrust at least 2 feet below the surface. The water surface of the tailrace was then 2.5 feet below the toe of the dam. This makes the bottom of the tailrace more than 9.5 feet below the top of the toe of dam. The tailrace near the dam passed over a shoal-like formation and entered the tailrace pond about 50 feet from the east end of the dam. Between this point and the narrow neck alluded to the tailrace was bounded by the toe of the dam and an elliptical shore line. Its maximum width was 125 feet, its narrowest width (at the neck) 45 feet. It was at this neck that many measurements of flow were made during 1888 and 1889 for the United States Geological Survey. In March, 1899, simultaneous measurements were made of the tailrace at this neck and of the forebay; and while the flow through the tailrace exceeded that in the forebay, the difference was not greater than could be accounted for by the fluctuations of flow caused by increased demands on the street-car service. Early in 1899, when the lake level was 10 feet below the crest of the dam, the writer urged some of the authorities to shut off all power some night at 12 o'clock, and after the tailrace water had run off to have the flow measured in order to ascertain the leakage, if any; but there were difficulties which prevented the stopping of the works. It is probable that there was no leak under the dam from the lake. Water from the lake going under the dam would have been under a head of 66 feet, and would have emerged with a velocity of

A. BROKEN DAM, SHOWING IN FOREGROUND SAND BAR LEFT IN MAIN CHANNEL.

B. REMNANTS OF DAM.

more than 64 feet per second, which would have cut away the limestone foundation in a few hours.

Mr. E. W. Groves, who was connected with the work, as an engineer, from the preliminary surveys to the completion of the dam, states that for the first 150 feet from the east bluff very good rock was found; that at that point a fault 75 feet wide was encountered, in which there was no semblance of stratified rock, most of the material being adobe or pulverized rock, with an occasional streak of red clay; that the excavation in this space was carried down 8 or 10 feet in the upstream trench, and the trench widened from 4 feet to 10 or 15 feet; that the fault extended to an indefinite depth; that from the west edge of the fault the foundation rock was poor for 350 feet, and that a supplementary protection was added to the upstream side opposite the fault by dumping clay along the face of the dam.

The limestone formation in the vicinity of the dam consisted of alternately hard and soft strata. The outcropping in Bee Creek (just above

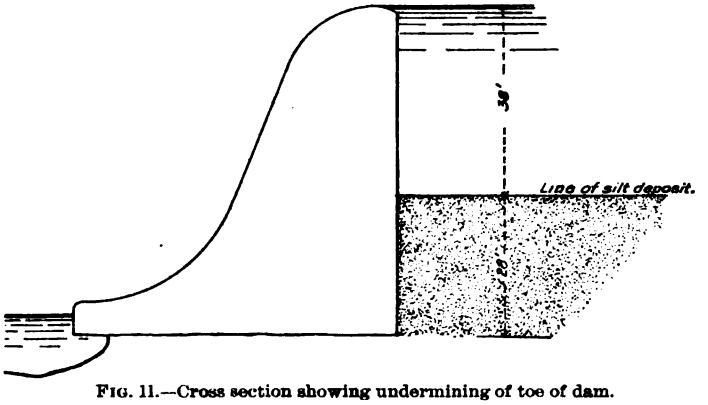


FIG. 11.—Cross section showing undermining of toe of dam.

the dam), that at Taylor's lime chute (about a half mile above the dam), and that through which the excavation was made to repair the head-gate masonry are all of that character. The soft strata could be handled without a pick and often with a shovel, but the hard strata was composed of a fairly good quality of limestone. In its western part the dam rested on one of these hard stratas. During a freshet in 1892 the overfall cut through this hard strata, tore up large pieces (some of them 10 feet long, 4 feet wide, and 2.5 feet thick and of 7 to 8 tons weight), and deposited a whole quarry in a confused and irregular pile about 150 to 200 yards farther down the river. These stones remained in that location until the big freshet of June 7, 1899, when they were carried away.

The foregoing facts are necessary for a proper understanding of what follows. In 1897, Mr. J. G. Palm, one of Austin's leading citizens and a cashier of the oldest national bank in the city, while fishing along the toe of the dam, ran his fishing pole under the toe for a

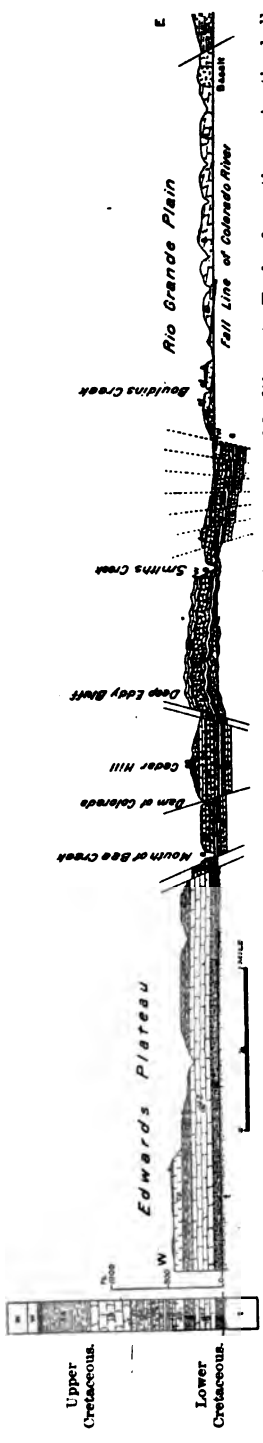


FIG. 12.—Geologic section east and west along Colorado River at Austin, Texas, showing Balcones zone of faulting. ta, Taylor formation; a, Austin chalk; ef, Eagle Ford shales; sc, Buda (Shoal Creek) limestone; dr, Del Rio clays; fw, Fort Worth limestone; e, Edwards limestone; cp, Comanche Peak limestone and Walnut formation; gr, Glen Rose formation; t, Travis Peak formation.

distance of 6 feet. This shows conclusively that either the water flowing along the tailrace had scoured out the foundation under the toe of the dam or the overfalling water had undermined it. A large percentage of the water flowing over the east half of the dam at ordinary height reenforced the tailrace waters and produced a strong current along the toe of the dam for more than half its length. It has now been proved, by actual measurements, that the toe was cut under at some place, as shown in fig. 11. In speaking about the matter Mr. Palm said that he often wondered why the toe did not break off. This undermining of the toe left the dam exposed to the pressure of the water, and it became only a question of which was the stronger—the water or the friction between the dam and its bed.

In regard to the geologic formations, the following is quoted from a letter of Mr. Robert T. Hill, of the United States Geological Survey, in the Engineering News of May 3, 1900:

> In the plateau country, which begins about a mile above the present site of the dam, the strata are firm and horizontal, and the river flows over ledges of firm and solid rock, which would have made a suitable and durable foundation for the construction. Just below this point, and within a belt of country upon which the dam is located, the strata are excessively jointed and faulted, constituting what is technically known as the Balcones fault zone, as shown in [fig. 12]. The geological formation is also different, consisting of the limestones of the Edwards formation, which are exceedingly porous and soluble, while to the west of the fault zone the strata are less soluble and more durable. The action of the subterranean waters upon the Edwards limestone results in dissolving it into caverns and crumbling strata, even where at the surface it appears perfectly solid and durable. Furthermore, artesian springs of great volume and pressure well up the joint planes and fissures in this formation. The site of the dam chosen crossed the river subparallel to one of the most conspicuous fault lines, at the northern (eastern) end of which, after the excavation and

construction had well advanced, a spring of the character mentioned developed, which greatly endangered the tie-on at the end and cost many thousands of dollars to circumvent. * * * Had the dam been located less than 2 miles above the present site, this structural condition would have been avoided.

A second geological consideration in the construction of the dam, and one to which sufficient attention was not, in my opinion, paid, was in the choice of material. Within 60 miles of Austin by rail are some of the most superb granite quarries in the world. This material was used to face the dam, but its center was built of the same soluble limestone as that previously mentioned, which was obtained from a quarry at the mouth of Bee Creek on the south (west) side of the river, less than half a mile from the dam. An examination of the face of the quarry shows the character of the material taken from it for use in the dam, and a glance is sufficient to show that its solubility was such as to render it utterly untrustworthy.

Mr. J. P. Frizell, after an examination of the preceding statements, adds that the location at Mormon Falls, 2 miles above the chosen site, presented points of decided superiority over the locality selected, but the board of public works thought that location inconsistent with the purposes of the improvement. Mr. Frizell does not consider that the solubility of the rock had any bearing on the failure, and sees no reason to doubt that the immediate cause was the undermining on the downstream side, caused by the abrasive action of the current and the constant stream of water coming from the power house and flowing along the toe of the dam on its way to the open channel of the river. A progressive weakening is attested by the fact that during the preceding year the dam had withstood a flood substantially as great as the one in which it failed. The toe of the dam, which was left without support by the undermining, contained granite blocks of more than 6 tons weight.

It is on record that the breaking down of this unsupported toe was imminent, in which event each of these stones would become a millstone (propelled in such a flood by some 2,000 horsepower) in the work of grinding the friable rock bottom and extending the undermining. At the wooden dam across Connecticut River at Holyoke, Massachusetts, an action of this kind became threatening in 1866. A pit 20 feet deep had formed on the downstream side of the dam. This danger was met by the construction of a massive apron of crib work filled with stone, which prolonged the duration of the structure more than thirty years, or until the construction of the present stone dam. At Austin the engineer had in contemplation from the beginning an analogous work, viz, an extension of the massive apron by a bed of concrete, to be applied as soon as the abrasive action had made sufficient progress to indicate the character and extent of the work required for its suppression.

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56TH CONGRESS, 2d Session. } HOUSE OF REPRESENTATIVES. { DOCUMENT No. 484.

DEPARTMENT OF THE INTERIOR

WATER-SUPPLY

AND

IRRIGATION PAPERS

OF THE

UNITED STATES GEOLOGICAL SURVEY

No. 41

THE WINDMILL: ITS EFFICIENCY AND ECONOMIC USE, PART I.—MURPHY

WASHINGTON
GOVERNMENT PRINTING OFFICE
1901

UNITED STATES GEOLOGICAL SURVEY

CHARLES D. WALCOTT, DIRECTOR

THE WINDMILL:

ITS EFFICIENCY AND ECONOMIC USE

PART I

By EDWARD CHARLES MURPHY

WASHINGTON
GOVERNMENT PRINTING OFFICE
1901

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LETTER OF TRANSMITTAL.

DEPARTMENT OF THE INTERIOR,
UNITED STATES GEOLOGICAL SURVEY,
DIVISION OF HYDROGRAPHY,
Washington, December 5, 1900.

SIR: I have the honor to transmit herewith a manuscript entitled The Windmill: Its Efficiency and Economic Use, Part I, by Edward Charles Murphy, formerly professor of civil engineering at the University of Kansas, and to request that it be published in the series of Water-Supply and Irrigation Papers.

This paper, with its complement, Paper No. 42, is a revision of Paper No. 8, printed in 1897, embodying additional data obtained by recent investigations. The demand for Paper No. 8 was so great and the practical applications of windmills to the problem of bringing water to the surface have been so extended that it has seemed desirable to make available the conclusions given in the following pages.

Very respectfully,

F. H. NEWELL,
Hydrographer in Charge.

Hon. CHARLES D. WALCOTT,
Director United States Geological Survey.

THE WINDMILL: ITS EFFICIENCY AND ECONOMIC USE.

PART I.

By EDWARD CHARLES MURPHY.

INTRODUCTION.

History does not record the name of the person who invented the windmill, nor give the date of the invention. The belief that windmills were used by the Romans is not well authenticated, and their use by the Bohemians in 718 is doubted. It is quite clear, however, that they were used in France and Italy in the twelfth century for grinding corn, and that they were used in Holland in the fifteenth century for pumping water over the dikes into the sea.

Mr. John Burnham, of Connecticut, is said to be the inventor of the American windmill. Mr. L. H. Wheeler, an Indian missionary, patented the Eclipse mill in 1867. The first steel mill was the Aermotor, invented by Mr. T. O. Perry in 1883.

The common European windmill, shown in section in figs. 1 and 2, differs much in appearance from the American mill. The wind wheel of the European mill has usually four long wooden arms, to each of which is attached a sail, against which the wind presses. The sails consist of a framework, on which canvas is stretched, usually forming a warped surface, the angle with the plane of the arms (called the angle of weather) being about 7° at the outer end and 18° at the inner end. The length of sail was usually about five-sixths the length of the arm, the width of the outer end one-third the length, and the width of the inner end one-fifth the length. The sail area is seen to be small compared with the wind area or zone containing the sails. The arms were sometimes 60 feet long. The American wooden mill is much smaller and more compact than the European mill. It has six or eight arms, to which is attached a framework carrying many small sails. These sails are usually 3 or 4 feet long, 3 or 4 inches wide at the outer end, and 1 to 3 inches wide at the inner end, and are set at an angle of 30° to 40° to the plane of the wheel. For large wheels these sails are arranged in two or more concentric rings. The Ameri-

can steel mill differs principally from the wooden mill in that it has larger and fewer sails for a given size of mill, its sails are curved instead of plane, and it offers less resistance to the passage of the air over the back of the sails. In the sails of the steel mill there is seen to be a partial return to those of the European mill.

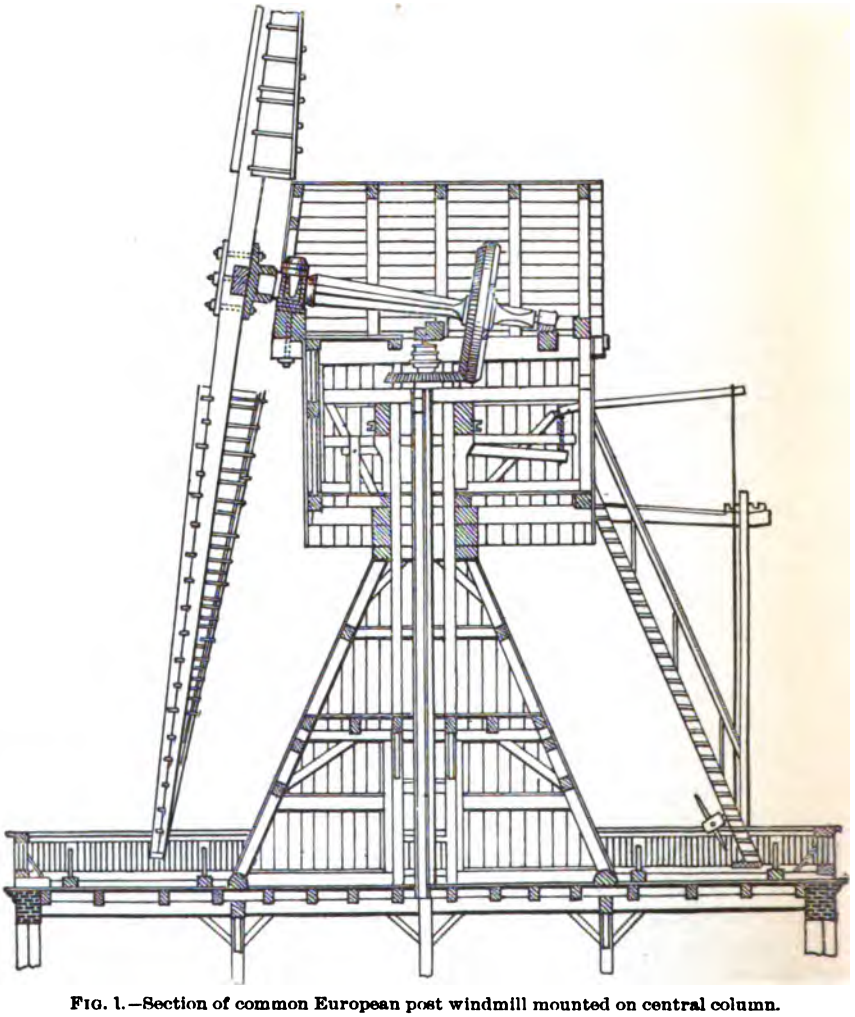


FIG. 1.—Section of common European post windmill mounted on central column.

CLASSIFICATION OF WINDMILLS.

Windmills may be divided into two general classes—paddle wheel and sail wheel. The Jumbo shown in Pl. XVI, *B* (Part II), and Little Giant No. 56, described on pages 125 to 127, Part II, are good illustrations of the first class. In both of these mills the sails move with the wind, and it is necessary to have a shield, or a method of feathering

the sails, in order to keep the wind from striking them when they are moving in a direction opposite to that of the wind. In the Jumbo the axis of the wheel is horizontal, in the Little Giant it is vertical; but the wind acts on the sails of both in substantially the same way. The air acts with full pressure on only one sail of the Jumbo at any one time, and on half the sails it has no action, or only negative action.

In the sail-wheel mill (fig. 1, Part I, and Pl. XV, Part II) the wheel moves at right angles to the direction of the wind, instead of in the same direction, as in the paddle-wheel mill. The wind acts with a

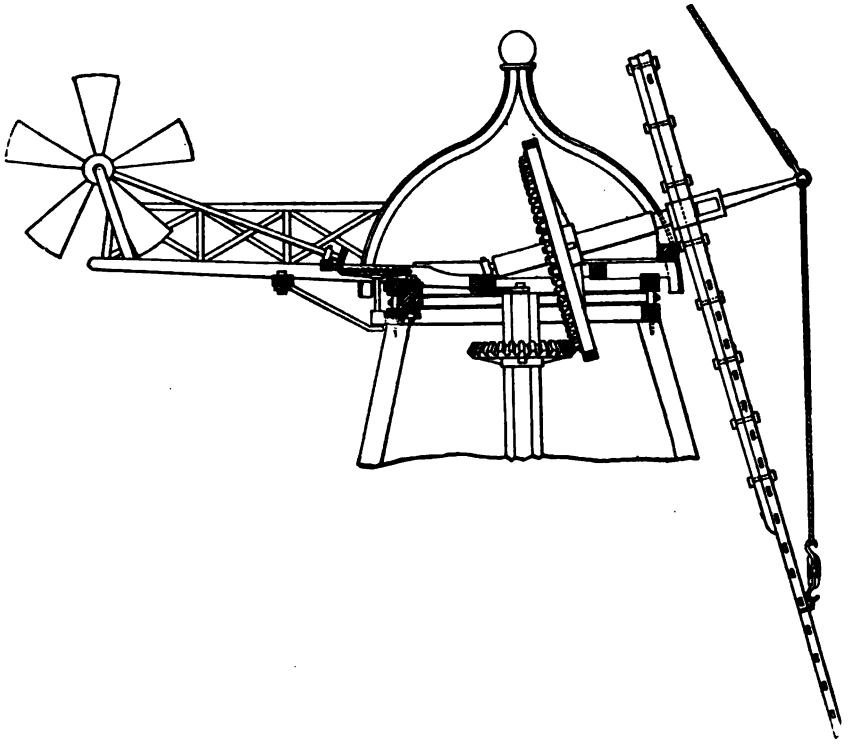


FIG. 2.—Early form of head of European tower windmill.

certain pressure on all of the sails all of the time. The circumference velocity of the sail wheel may be two or more times greater than the velocity of the wind that drives it, but the circumference velocity of the paddle wheel is always less than the velocity of the wind that drives it. The sails of the sail-wheel mill must be placed at an angle with the plane of the wheel, so that the wind will press on them; but the sails of the paddle-wheel mill may be in the plane of the axis of the wheel.

For greatest pressure on the sails of a sail-wheel mill the axis of the

wheel must be parallel to the direction of the wind. It is necessary, then, for the wheel to change its direction as the wind changes. In the German or post mill the whole building, as well as the wheel, can be turned around on a post by hand. In the tower or Dutch mill the upper part only of the mill turns with the wheel. This is accomplished by hand in the mill shown in fig. 1, and by an auxiliary windmill in the mill shown in fig. 2. In the American mills the upper part only turns on a turntable, which is usually on rollers or balls. This is accomplished, first, by the pressure of the wind on a long rudder vane extending out behind the mill, and in some mills by a side vane as well; or, second, by the pressure of the wind on the wheel itself, which is placed on the opposite side of the tower, as in fig. 46, Part II; or, third, by side wheels, as in mill No. 52, fig. 50, Part II.

REGULATING DEVICES.

The wind is constantly changing in velocity as well as in direction, and if the load on the mill is constant the speed of the mill and of the machine which it operates will change with it. If the speed is to be kept nearly constant, some device is needed to reduce the wind pressure on the wheel when the wind velocity reaches a certain amount. In the European mill there are two methods of doing this, viz, by means of a brake or friction rings and by changing the sail area. The latter is accomplished by rolling or unrolling the canvas sails by hand or automatically. In the American mill the speed is regulated by changing the sail area in one of three ways: (*a*) By sections of the wheel revolving about an axis which places each sail at an angle to the direction of the wind less than 90°, as in fig. 36, Part II; (*b*) by placing the axis of the wind wheel eccentric to the axis of the tower, so that the wind pressure on the wheel will cause it to revolve around the axis of the tower; and (*c*) by the wheel moving naturally around the axis of the tower in the direction in which it is revolving. This turning action around the axis of the tower is counteracted by a spring or a weight, or, as we commonly say, "the wheel is held in the wind" by a spring or a weight. A weight is better than a spring, for by moving it out or in any desired pull can be placed on the wheel. A spring can not easily be adjusted and may lose some of its tension. The first method of regulation, called the centrifugal-governor method, is used in the Halliday mill (fig. 20) and in the Althouse-Wheeler mill (fig. 36, Part II). The second method is illustrated in the Aermotor (fig. 33, Part II), which shows the axis of the wheel eccentric to the axis of the tower by 4.5 inches. It also shows the spring which holds it in the wind. This spring also resists the action of the load, which tends to turn the wind wheel out of the wind. The third method is illustrated by the Woodmanse mill.

DUTCH WINDMILL AT LAWRENCE, KANSAS.

EARLY EXPERIMENTS.

Smeaton's experiments.—The first experiments of which we have any record were made by John Smeaton, an English engineer, and published in 1755 to 1763.[1] These experiments were made on model windmills of the European type, for the purpose of determining the best shape of sail for a given sail area. The models each had four arms 21 inches long. For one set of tests the sails were 5.6 inches broad and 18 inches long, giving an area of 403 square inches. In another set of tests the sails were 18 inches long, 5.6 inches wide at the inner end and 8.4 inches wide at the outer end, with an area of 504 square inches. The sails were either plane or warped at various angles. These mills were worked by moving the windmill around in a circle of 5½ feet radius, in still air, instead of placing them on a tower and allowing the natural wind to drive them. The wheel was moved around in this circle by means of a cord wound on a drum on a vertical shaft, the horizontal arm which held the wheel being fastened to this shaft. The work done by the wheel in a given time was measured by observing the length of string wound on the shaft of the wheel, a weight of known size being attached to the end of the string. The velocity of the wind, which was assumed to be the velocity of the end of the arm where the wheel was attached to it, varied from 4⅓ to 8¾ feet a second, or from 2.9 to 6 miles an hour.

It will be noticed that these wheels are only 3.5 feet in diameter, that they were moved in a circle only 5.5 feet in diameter, and that wind velocities or wheel velocities were small and of only a limited range—from about 3 to 6 miles an hour. Smeaton draws the following conclusions from his experiments:

(1) The velocity of the windmill sails, whether unloaded, or loaded so as to produce a maximum, is nearly as the velocity of the wind: their shape and position being the same.

(2) The load at the maximum is nearly, but somewhat less than as the square of the velocity of the wind: the shape and position of the sails being the same.

(3) The effects of the same sails at a maximum are nearly but somewhat less than as the cubes of the velocity of the wind.

(4) The load of the same sails at the maximum is nearly as their squares and their effects as the cubes of their number of turns in a given time.

(5) When the sails are loaded so as to produce a maximum at a given velocity of the wind, and the velocity of the wind increases, the load remaining the same: first, the increase of effect, when the increase of the velocity of the wind is small, will be nearly as the squares of those velocities; secondly, when the velocity of the wind is doubled, the effects will be nearly as 10 to 27.5; but, thirdly, when the velocities compared are more than double of that where the given load produces a maximum, the effects increase nearly in a simple ratio of the velocity of the wind.

(6) If sails are of similar figure and position, the number of turns in a given time will be reciprocally as the radius or length of the sail.

[1] Philos. Trans. Royal Soc. London, 1755-1763.

(7) The load at a maximum that sails of a similar figure and position will overcome at a given distance from the centre of motion, will be as the cube of the radius.

(8) The effects of sails of similar figure and position are as the square of the radius.

(9) The velocity of the extremity of Dutch sails, as well as of enlarged sails, in all their usual positions, when unloaded, or loaded to a maximum, is considerably quicker than the velocity of the wind.

Regarding the ratio of the sail area to the wind area or zone containing the sails, he found that where the ratio was greater than 7 to 8 the power of the mill was decreased instead of increased. Regarding the proper shape of sail, he found that the warped sail was more effective than the plane sail. He also found the following six angles of weather at equal distances from the shaft outward advantageous: 70°, 71°, 72°, 74°, 77.5°, and 83°. He states that a difference of two or three degrees in the angles of impact makes little difference in the power of the mill.

Coulomb's experiments.—C. A. Coulomb, a French engineer, made some tests of the work done by a Dutch windmill used for extracting oil from rape seed at Lille, in Flanders. His observations were published in 1821.[1] The mill was 70.2 feet in diameter. It had four warped canvas sails, each 28.7 feet long and 6.6 feet wide; the width of canvas was 5.5 feet. The angle which the plane of sail made to the plane of the wheel varied from 30° at the inner end to 12° at the outer end. The wind velocity was measured by the use of feathers carried along by the wind. Two men were stationed 150 feet apart, on slight elevations, to note the time required for each feather to pass over this distance. The velocity of the wind striking the wind wheel was assumed to be that found from these feathers. In a 14.9-mile wind, and with the load ordinarily used, he found that the wheel made 13 revolutions per minute with all the canvas spread. From these data he figured the useful work done by the mill per minute to be 232,388 foot-pounds and the useless work expended in shock of stampers and friction to be 37,310 foot-pounds, or a total of 269,698 foot-pounds, equal to 8.17 horsepower.

Coulomb did not consider this a complete or satisfactory test of this mill. He did not control the working of the mill, but simply observed what it did when handled by the miller who extracted the oil. He tried to induce the owner to permit him to use the mill for a time for experimental purposes, but did not succeed.

It will be seen from what follows that even if we assume the wind velocity to be correctly measured, this test does not necessarily show the power of the mill, for we do not know that the load used was the proper load for the wind velocity. It shows what the mill was doing, not what it might do under a better loading.

[1] Theorie de Machines Simple, by C. A. Coulomb. Paris, 1821.

Griffiths's experiments.[1]—In 1891–92 Mr. J. A. Griffiths made tests of six windmills used for raising water, with the following results:

Results of windmill tests by J. A. Griffiths.

No.	Type of windmill tested.	Outer diameter of sail.	Inner diameter of sail.	Gross area of sail wheel.	Weather angle at outer end of sail.		Diameter of pump.	Stroke of pump.	Average head of water.	Load on pump, per stroke.a	Velocity of wind per hour.b	Velocity of mill per minute.b	Horsepower.b
		Feet.	*Feet.*	*Sq. ft.*	°	′	*In.*	*In.*	*Feet.*	*Ft.-lbs.*	*Miles.*	*Rev.*	
1	Toowoomba	22.3	8.3	39.20	18	47	5	6.75	25 100	120 480	4.3 7.0	5.0 6.8	0.018 0.098
2	Stover	11.5	4.5	104.0	43		3	4	29.2 61.2	29.2 61.2	5.8 6.5	13.0 13.3	0.011 0.025
3	Perkins	16.0	6.0	201.0	36		3	10.75	39.0	105.0	6.0	7.5	0.024
4	Althouse-Wheeler	14.2	4.5	157.0	30		3	c10	66.3	166.0	7.0	12.6	0.065
5	do	10.2	3.8	81.0	28		3	4½	38.7	51.0	8.5	20.5	0.028
6	Carlyle	9.8	4.2	80.0	50		3	4	30.7	30.7	6.0	12.5	0.012

a These pump loads have been computed for comparison of these results with others.
b At maximum efficiency.
c Pump is double acting; this is twice the length of stroke.

No. 1 was a 22.5-foot wooden mill, with 20 warped sails each 87 by 36 by 9 inches, the weather angle at the outer end being 18° 47′ and at the inner end 40° 20′. It worked a direct-acting single-stroke pump having a 5-inch cylinder and 6.75 inches stroke. Two lifts were used, one of 25 feet and the other of 100 feet.

No. 2 was a 12-foot Stover wooden mill, having a wind wheel somewhat like that of mill No. 38. It had 112 sails, each 43 by 3.75 by 1.5 inches, set at an angle of 43° to the plane of the wheel. It worked a direct-acting single-stroke pump having a 3-inch cylinder and 4 inches stroke. The lifts were 29.2 and 61.2 feet.

No. 3 was a 16-foot Perkins solid wood wheel, the wind wheel having 160 sails, each 60 by 4 by 1.5 inches, set at an angle of 36° to the plane of the wheel. It worked a direct-acting double-stroke pump having a 3-inch cylinder and 5.375 inches stroke. The lift was 39 feet.

No. 4 was a 14-foot Althouse-Wheeler wooden sectional mill, the wind wheel having 104 sails, each 48 by 4 by 1.5 inches, set at an angle of 30° to the plane of the wheel. It worked a direct-acting double-stroke pump having a 3-inch cylinder and 5 inches stroke. The lift was 66.3 feet.

No. 5 was a 10-foot Althouse-Wheeler wooden sectional mill. The wind wheel had 84 sails, each 38 by 3.75 by 1.5 inches, set at an angle of 28° to the plane of the wheel. It worked a direct-acting single-stroke pump having a 3-inch cylinder and 4.625 inches stroke. The lift was 38.7 feet.

No. 6 was a 10-foot Carlyle iron mill. It had 7 somewhat spoon-shaped sails, each having a spout-like extension through which the

[1] Windmills for raising water, by J. A. Griffiths: Proc. Inst. Civ. Eng., Vol. CXIX, p. 321.

air flowed. It worked a direct-acting single-stroke pump having a 3-inch cylinder and 4 inches stroke. The lift was 30.7 feet.

The wind velocity was measured with the "f" wind gage, which was either on a tower near by or on an arm projecting as far as possible to windward. It appears that it was necessary to be near the gage in order to read the velocity, which would indicate a possible error in wind velocity and some interference with the wind striking the wind wheel. The range of wind velocities is not stated, and not more than two loads were used in any case.

These results will be compared with others further on.

King's experiments.[1]—Prof. F. H. King conducted a series of experiments with a 16-foot geared Aermotor, covering a period of one year—from March 6, 1897, to March 6, 1898. This mill was used to work one or more of four pumps: (1) A reciprocating piston pump with a 14-inch cylinder and 9 inches stroke; (2) a bucket pump having a normal capacity of 120 gallons per minute; (3) a No. 2 Gould centrifugal pump; and (4) the smallest size Menge pattern centrifugal pump. The bucket pump was used nearly all of the time. The reciprocating piston pump was used occasionally by itself and part of the time with the bucket pump, when the wind velocity was strong enough to carry both. The Menge was used occasionally with the piston pump and the bucket pump, when the wind velocity was strong enough to carry all.

There was no automatic device for throwing into or out of use any of these pumps. It was necessary to do this by hand, so that a part of the time the load on the mill was not suited to the wind velocity. This can be seen from the record. For example, on February 10, from 1 to 7 p. m., the wind velocity varied from 9 to 12 miles an hour and 8.6 tankfuls of water were pumped, while on June 1, from 8 a. m. to 4 p. m., the wind velocity varied from 11 to 15 miles an hour, and not a tankful was pumped. The report gives the number of tankfuls of 141.2 cubic feet which were lifted 12.85 feet each hour during the year, and some interesting conclusions drawn from these records.

Professor King has also made some tests of this mill with a Prony friction brake. The results of these tests, and the indicated horsepowers computed from them, are as follows:

[1] Bull. No. 68, Wisconsin Agricultural Experiment Station.

Results of tests of 16-foot geared Aermotor.

Wind velocity per hour.	Direction of wind.	Indicated horse-power.	Average wind velocity.	Average horse-power.	Barometer.	Temperature.
Miles.			*Miles.*		*Inches.*	*Degrees.*
8.4	SW.	0.2715	8.4	0.2715	29.40	0.0
12.0	----------	0.5791	12.33	0.5858	29.36	0.5
12.4	----------	0.7230			29.36	0.5
12.6	----------	0.4553			29.36	0.25
13.2	SW.	0.6213	13.2	0.6213	29.40	0.5
14.6	SW.	0.7343	14.68	0.8602	29.36	−0.5
14.6	----------	0.9449			29.40	0.5
14.8	SW.	0.9016			29.40	1.0
18.6	NW.	2.054	18.70	1.873	28.86	9.0
18.8	SW.	1.692			29.40	1.0
21.2	SW.	2.593	21.55	3.033	29.40	1.5
21.6	W.	3.715			29.06	−2.0
21.6	N.	3.227			28.86	7.0
21.8	NW.	2.597			-------	----------
22.0	W.	4.326	22.06	3.652	29.09	1.0
22.0	W.	4.236			29.04	−2.0
22.2	NW.	2.394			-------	----------
23.0	W.	3.842	23.00	3.996	29.07	−1.0
23.0	W.	4.151			29.05	−1.0
24.0	----------	5.983	24.30	4.768	28.75	6.8
24.6	W.	3.554			29.09	1.5
25.2	W.	4.882	25.2	4.882	29.09	1.5
27.3	N.	4.092	27.16	4.471	28.66	6.3
27.0	W.	4.850			29.10	1.5
39.0	N.	5.953	39.0	5.953	28.69	6.5
40.0	----------	5.971	40.0	5.971	28.69	6.3

This mill is similar to our 16-foot Aermotor No. 44, described in Part II. It will be seen later that the power found by Professor King is much greater than we have found it for high wind velocities. The probable reason for this difference will be discussed later. It may be stated here, however, that Professor King measured the wind velocity with an anemometer in a fixed position 40 feet due east of the windmill, and it will be seen that the wind was from the west, northwest, or southwest nearly all of the time when the brake tests were being made. (The wind came from the north around by the east to the south only three times while the brake tests were in progress.) The revolving wheel must have interfered with the running of the anemometer and caused it to show a less wind velocity than really existed.

Professor King also determined the work done by the mill in grinding corn. The power of the mill in a given wind velocity can not, however, be judged from these tests, since the grinder load was probably not suited to all wind velocities.

Results of brake tests of 16-foot geared Aermotor.

Wind velocity per hour.	Indicated horse-power.	Wind velocity per hour.	Indicated horse-power.
8 miles	0.25	26 miles	4.82
10 miles	0.40	28 miles	5.14
12 miles	0.56	30 miles	5.40
14 miles	0.78	32 miles	5.61
16 miles	1.08	34 miles	5.76
18 miles	1.62	36 miles	5.87
20 miles	2.39	38 miles	5.95
22 miles	3.31	40 miles	5.97
24 miles	4.31		

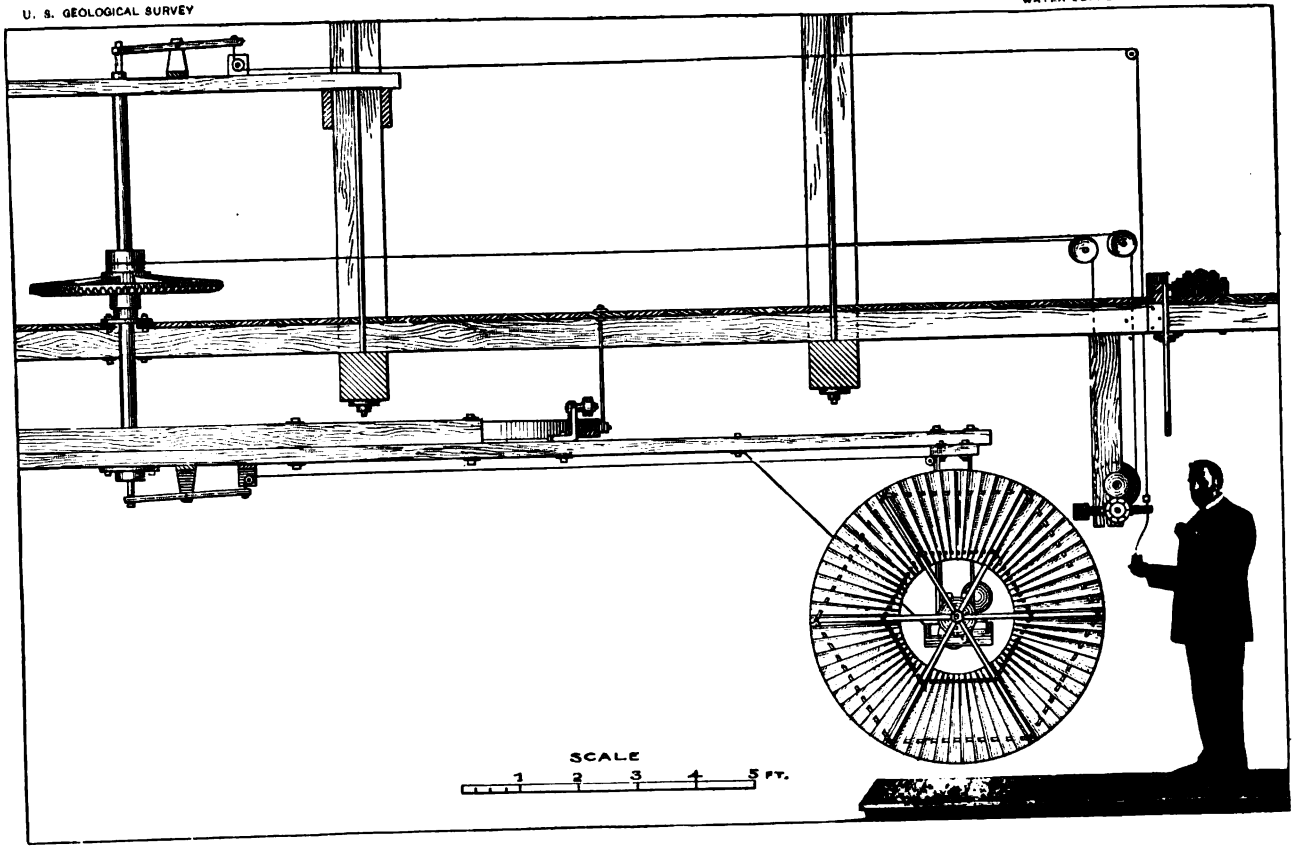
Perry's experiments.—From June, 1882, to September, 1883, Mr. T. O. Perry made experiments with 61 windmills, each 5 feet in diameter. The results were published in 1889.[1] Mr. Perry's methods were similar to those employed by Smeaton—he used small wheels moved against still air in a circle of 14 feet radius. His experiments were made on a much larger scale than Smeaton's, however, and his apparatus was more perfect. Smeaton used wheels of European type; Mr. Perry used those of American type. Pl. II is an elevation of Mr. Perry's apparatus, showing the wheel as it revolved about the vertical shaft, driven by an 80-horsepower engine. The power was measured by means of a Prony friction brake placed on a brass cylinder on the wind-wheel shaft. In order to eliminate the effect due to differences in the condition of the air, and get results comparable with one another, Mr. Perry used one of his wheels as a standard with which to compare the others. After the best load for a wheel had been obtained, comparative tests were made with this one and with the standard wheel, by trying first one wheel and then the other until several measurements of each had been taken. The final result of each wheel was the average of eight or ten measurements.

In comparing Smeaton's results with his own, Mr. Perry writes:

> We were not able to obtain the best results with weather angles as small as Smeaton's in any of our wheels. Nor did our sail speeds, as compared with wind velocity, nearly approach the speeds obtained by Smeaton. Even our unloaded wheels did not show the sail speed attained by the best of Smeaton's when loaded for maximum work. * * * Our loads at the maximum of work were smaller as compared with the greatest loads, and the speed of revolutions at maximum work as compared with the speeds of unloaded wheels, were smaller for our mills than for Smeaton's.

He states, however, that the general conclusions drawn by Smeaton (see pages 15 to 16) were substantially confirmed by his experiments. The difference between his results and Smeaton's he attributes to the differences in the mills used.

[1] Water-Supply and Irrigation Paper U. S. Geol. Survey No. 20.



ELEVATION OF APPARATUS USED BY PERRY IN WIND-WHEEL TESTS.

Some of Mr. Perry's conclusions are as follows:

(1) There is nothing gained by having the sail area more than seven-eighths of the wind area, and there is little gained by having it more than three-fourths of the latter area.

(2) That the power varies as the cube of the wind velocity.

(3) That the load for maximum power varies as the square of the wind velocity.

(4) That the speed of the unloaded wheel increases somewhat faster than the wind velocity.

(5) That the best speed for most of his wheels was about 0.55 per cent of the unloaded speed.

(6) That the conical deflector at the center of the wheel does not increase the power.

(7) That obstructions on the back of sails greatly reduce the power of the mill.

(8) That the speed of wheel No. 48 was increased 48 per cent by removing the strip from the back of each sail.

(9) That a deflector in front of the wheel increased the speed of a slow-moving wheel.

(10) That a deflector in front of the wheel did not increase the speed of a rapidly moving wheel.

(11) That a mast offers more obstruction in front of a wheel than behind it.

EXPERIMENTS BY WRITER.

The tests of windwills described in the following pages were begun by the writer in the summer of 1895. They were continued during the summer of 1896, with much better facilities than during the previous season. The results obtained to that time were published in Water-Supply and Irrigation Paper No. 8, entitled Windmills for Irrigation. Since then the work has been continued as time could be spared—mainly during a portion of three summer vacations. The work of the summer of 1896 was confined mainly to pumping mills. The tests show what each windmill and its pump were actually doing under certain conditions of load, lift, etc. They do not show what the mill might do under other conditions. It was evident that the useful work of a mill varied with its load and the efficiency of the pump. The latter could not well be ascertained. It was, therefore, thought best to confine the tests principally to power mills, in which the unknown factor of pump efficiency is not present, and where the load on the mill can easily be varied. This has enlarged the scope of the work, making it cover windmills for power as well as those for irrigation.

Many of our tests of pumping mills were made in the vicinity of Garden, Kansas. Perhaps nowhere in the United States is irrigation from wells by the use of windmills carried to the same extent as there,

where may be found hundreds of windmill pumping plants furnishing water to irrigate from 1 to 15 acres each and lifting from 3 to 14.5 quarts per stroke to a height of from 10 to 45 feet, as well as large steel mills running day and night, when the wind is strong enough, and working pumps of the best kind.

SCOPE OF TESTS.

There are many makers of American windmills, and with the great variety of mills in use—no two are alike, though in some cases the difference is slight—it was impossible to test a mill of each type. It was our purpose to test only the mills that were in good working order and subject to good wind exposure, and which would add new data to that already obtained or confirm in some particular that previously secured. In some cases two or more mills of the same size and make were tested to show as far as possible the effect of pump, well, etc. on the useful work. The parts of mill and pump on which the power depends were carefully measured. The temperature and barometric pressure were observed in each case and the mean given. The discharge of pump per stroke was measured when possible, and the diameter of cylinder and length of stroke are given, so that the discharge can be compared with the figured displacement. The lift was measured whenever the surface of the water in the well could be reached with a tapeline. The number of strokes of the pump per mile of wind was found for velocities from 6 or 8 miles to 20 or 30 miles an hour. In some cases the number of strokes is given when no water was being pumped. In fact, there was collected for each mill as much data as it was conveniently possible to obtain which would in any way affect the power of the mill or be of interest.

PUMPING MILLS.

The essential difference between pumping and power mills is that in the former there is a pump rod with an up and down motion, while in the latter there is a vertical rotating shaft. The former is usually geared back 2 or 3 to 1, while the latter is generally geared forward 6 or 8 to 1. The ordinary pumping mill, such as is used for stock purposes, is lighter than the power mill, but the irrigating mill is of nearly the same weight as the power mill. The larger power mills have a pumping attachment, so as to work a pump as well as a grinder or other machine.

WELLS NEAR GARDEN, KANSAS.

A brief description of the water supply and wells of this locality may be helpful in considering what follows.

The water is found in sand and gravel at distances below the surface varying from 8 to 40 feet. This material is in layers of variable thickness and different degrees of coarseness, ranging from fine sand

to large gravel. It is overlain by a layer of sandy clay, which in some places will for years stand vertical without any support; in other places there is very little clay in this layer. The wells are usually 3 to 4 feet square, and are cased with wood through the top sandy clay to the water-bearing sand; then a wood or galvanized-iron casing from 12 inches to 3 feet in diameter extends down from 8 to 20 feet into the sand to a layer of gravel. Where this latter casing is large, three or more galvanized-iron pipes 6 to 12 inches in diameter are put down in the bottom of it, and these sometimes have wire gauze over their tops to keep down the sand; they also have perforations about one-fourth of an inch in diameter for a distance of 2 feet or more from the bottom to admit the water. In many cases, instead of this small open well, the supply pipe is on a well point having the same diameter as the supply pipe, its length varying with the diameter. These well points have not given satisfaction and are being replaced by open wells.

On examining well points that have been used for a time it was found that many of the little openings through which water is admitted to the pump had become filled with fine grains of sand, thus reducing the area. Although this water area was of the proper amount when the well was new, it becomes too small after the well has been used for a time or after it has stood without being used. If this area is too small to allow the free passage of water into the pump, an added load is put on the latter.

PUMPS.

Nearly all of the pumps in use in the vicinity of Garden are of the reciprocating-piston type. Fig. 8 shows the Stone pump, manufactured by R. G. Stone, of Garden. This pump is made in three sizes—6 inch, 8 inch, and 10 inch, these dimensions being the approximate diameter of the discharge pipe. The diameter of the cylinder is less than the diameter of the pipe by twice the thickness of the brass lining. The valves (shown in fig. 9) are of the latest pattern. The plunger valve is of the single-flap or clack variety and the check valve is of the disc variety, made so that the water can pass up the center as well as around the sides. In an earlier form of this pump the plunger valve is of the double-flap or butterfly type and the check valve of the lift type, but with no opening at the center. Probably nine-tenths of the pumps in use near Garden are of the Stone variety.

Fig. 3 shows the Gause pump, one of the first pumps used there for irrigating purposes. It is more expensive than the Stone pump, and is not now so much used. Fig. 16 shows the cylinder of an 8-inch Frizell pump, a few of which are in use. Fig. 5 is a sectional view of the Woodmanse pump, which is used with mill No. 2. Pl. X, *B*, shows a crude homemade pump called the "water elevator." One of these is in use in Garden.

The efficiency of reciprocating pumps like those described varies

directly with the lift, inversely with the number of strokes per minute, and with the design of the pump. For lifts of 10 or more feet and not more than 30 strokes per minute the efficiency in a good pump should be at least 70 per cent.

Prof. O. P. Hood[1] has measured the efficiency of two Frizell pumps—one 6 inch, with 14.1 inches stroke, like that shown in fig. 16, and one 4 inch, with 24 inches stroke, having a butterfly discharge valve. He found that for a 7.7-foot lift the efficiency of the 6-inch pump dropped from 75 per cent to 63 per cent as the number of strokes increased from 10 to 60 per minute; that for a 22.7-foot lift it decreased from 86 per cent to 82 per cent for the same range of speed; and that for a 37.8-foot lift it decreased from 84 per cent to 82 per cent, while the number of strokes increased from 10 to 50 per minute. The efficiency of the 4-inch pump dropped from 66 per cent to 60 per cent for a 12.8-foot lift, and from 83 per cent to 73 per cent for a 37.6-foot lift, as the number of strokes increased from 10 to 50 per minute.

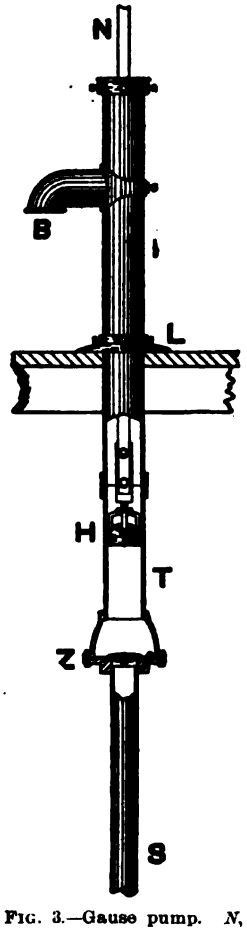


FIG. 3.—Gause pump. *N*, plunger; *B*, spout; *F*, discharge pipe; *H*, plunger; *T*, cylinder; *Z*, enlarged valve opening and check valve; *S*, suction pipe.

The valve area should be not less than 30 per cent of the cylinder area. The cylinder should be placed as near the water as possible; if it is more than 25 feet above the water, and the number of strokes is 30 or more, the cylinder will not fill properly, and pounding will result.

INSTRUMENTS AND METHODS.

The wind velocity was measured with a United States Weather Bureau cup anemometer, each mile of wind being recorded electrically by one pen of a 2-pen register. By means of a little device fastened to the pump an electric circuit is closed at each stroke of the pump and a record made by a recorder. Another electric circuit, leading from the recorder to the other pen of the register, is closed at each hundred strokes of the pump and a record made on the register. Hence the graphic record of the register shows the number of miles of wind in any given time, also the number of hundred strokes of the pump in the same time. The anemometer was held on a pole at the height of the axis of the wheel of the windmill. The pole was made so that its length could be increased at will from 25 to 50 feet. The anemometer on the pole is shown in Pls. III and XI.

[1] Water-Supply and Irrigation Paper U. S. Geol. Survey No. 14.

The discharge of the pump per stroke was ascertained by catching the water for several strokes in a tub and measuring it with a quart measure. In a few cases it was found to vary with the number of strokes per minute. Where it varied the discharge given is for a nearly maximum speed of pump.

The lift, or distance from the surface of the water in the well to the center of the water column as it leaves the discharge pipe, was measured when the pump was working quite rapidly. For pumps on well points it was estimated from the depth to water when the point was put down, making an allowance for the lowering of the water.

Each mill tested is described and the results of the tests given in tabular form. Nearly all of the mills are illustrated.

The number of strokes of the pumps per mile of wind and the horsepowers of the mills are in most cases explained by diagrams, which show at a glance the facts which otherwise can be comprehended only by a careful analysis of the tables. In these diagrams (figs. 6, 10, 11, 12, 13, 15, 17, 18, 19, and 21) the relation between the wind movement, in miles per hour, and the number of strokes made by the pump while the wind was moving over 1 mile is shown by the curved line. The space from left to right is proportional to the number of strokes of the pump. The data expressed by these diagrams were obtained directly from the record given by the anemometer register.

In explanation we will assume that the pen connected with the anemometer makes three short marks (3 miles) in fifteen minutes, indicating a mile in five minutes, or at the rate of 12 miles an hour. At the same time the other pen connected with the pump, and registering each 100 strokes, makes, say, two short marks, showing that the pump has made 200 strokes for this 3 miles of wind movement, or 67 strokes to the mile. This fact is entered on the diagram by a small circle placed at a distance from the right which corresponds to a wind velocity of 12 miles an hour, and at a distance from the bottom which corresponds to 67 strokes of the pump. In this way each observation is indicated. When the points have been plotted, the smooth curve is sketched so as to occupy an intermediate position among them.

In order to obtain the number of strokes more accurately than by measurement on the register sheet, they were actually counted for a considerable number of observations in each test. The number of strokes per minute is obtained by dividing the number of strokes per mile of wind by the number of minutes required to make the mile. For example: If the number of strokes per mile in a 12-mile wind (which requires $60 \div 12$, or five minutes to make a mile) is 90, then $90 \div 5 = 18$, the number of strokes per minute. The number of gallons raised per minute is found by multiplying the number of gallons

per stroke by the number of strokes per minute. The horsepower of the mill in any wind velocity is found by multiplying the number of

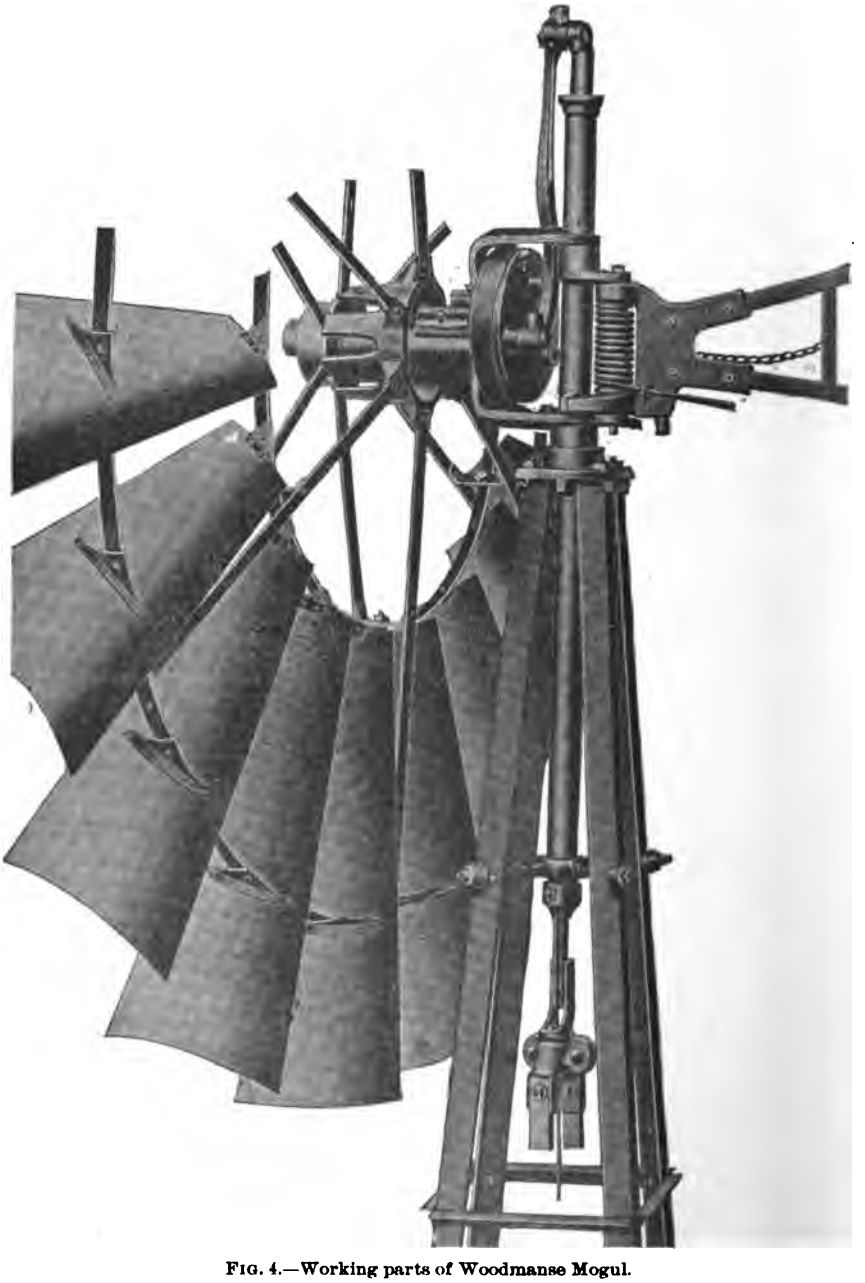


FIG. 4.—Working parts of Woodmanse Mogul.

gallons per stroke by the number of strokes per minute, then by 8.3 pounds (the weight of one gallon of water), then by the lift, in feet,

and dividing the product by 33,000 (the number of foot-pounds in a horsepower), or by the following formula:

Horsepower $= nqgh \div 33{,}000$, where $n =$ number of strokes per minute, $g =$ the weight of one gallon of water, $q =$ the number of gallons per stroke of pump, $h =$ the lift, in feet. The pump load is the weight of water lifted per stroke multiplied by the lift, or height to which it is raised. The number of revolutions of the wind wheel per minute is found by multiplying the number of strokes per minute by the number of revolutions per stroke.

FIG. 5.—Sectional view of Woodmanse pump.

PUMPING MILLS TESTED.

Mill No. 1.—The tests of this mill were preliminary or experimental, being made for the purpose of perfecting the instruments employed, and were not completed for discussion.

Mill No. 2.—This is a 12-foot Woodmanse Mogul, manufactured by the Woodmanse-Hewitt Manufacturing Company, of Freeport, Illinois. Pl. III shows the mill, tower, pump, and pond, and fig. 4 the working parts. The tower is of steel, 50 feet high to the axis of the wheel. The wind exposure on the north is not good, the mill being 115 feet south of a large barn. The wheel has 30 curved sails, each 36 by 13 by 5.5 inches,[1] set at an angle of 30° (angle of weather) with the plane of the wheel. It is back-geared, 3 to 1, and held in the wind by a spring. The pump

[1] In this expression 36 is the length of the sail, 13 the width of sail at the outer end, and 5.5 the width of sail at the inner end.

also is of Woodmanse make, and is shown, in section, in fig. 5. The cylinder is 9.5 inches in diameter, the supply pipe 5.625 inches in diameter, the length of stroke 12 inches. The well is 3¾ feet by 3¾ feet to the water, a distance of 14 feet. At that point a 12-inch galvanized-iron pipe is put down 20 feet, forming a small open well. The lift at the time of test was 17¾ feet and the discharge per stroke 14½ quarts. The mean barometric pressure was 26.98 inches, and the mean temperature 94° F. The cost of mill, tower, pump, and well was about $210. The results of the tests are as follows:

Results of tests of mill No. 2—12-foot Woodmanse Mogul.

[Load per stroke, 536.2 foot-pounds.]

Wind velocity per hour.	Revolutions of wind wheel per minute.	Strokes of pump per minute.	Gallons pumped per minute.	Useful horsepower.
12 miles	15.6	5.2	18.8	0.085
16 miles	48.0	16.0	58.0	0.260
20 miles	60.9	20.3	73.6	0.322
25 miles	69.9	23.3	84.5	0.379
30 miles	75.9	25.3	91.7	0.411

The curve shown in fig. 6 is for a moderately loaded 12-foot mill (536.2 foot-pounds per stroke). It starts at a wind velocity of 11

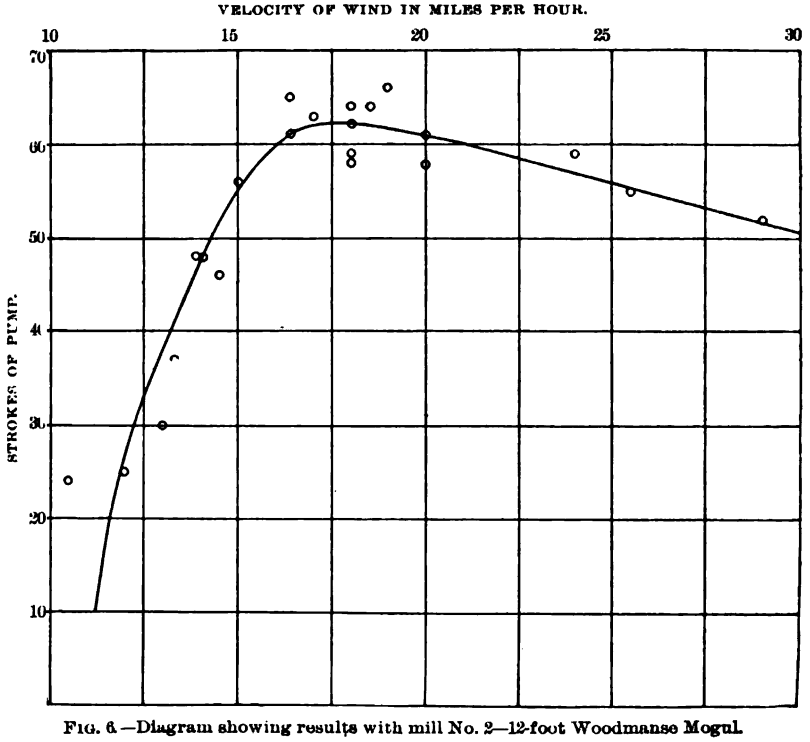
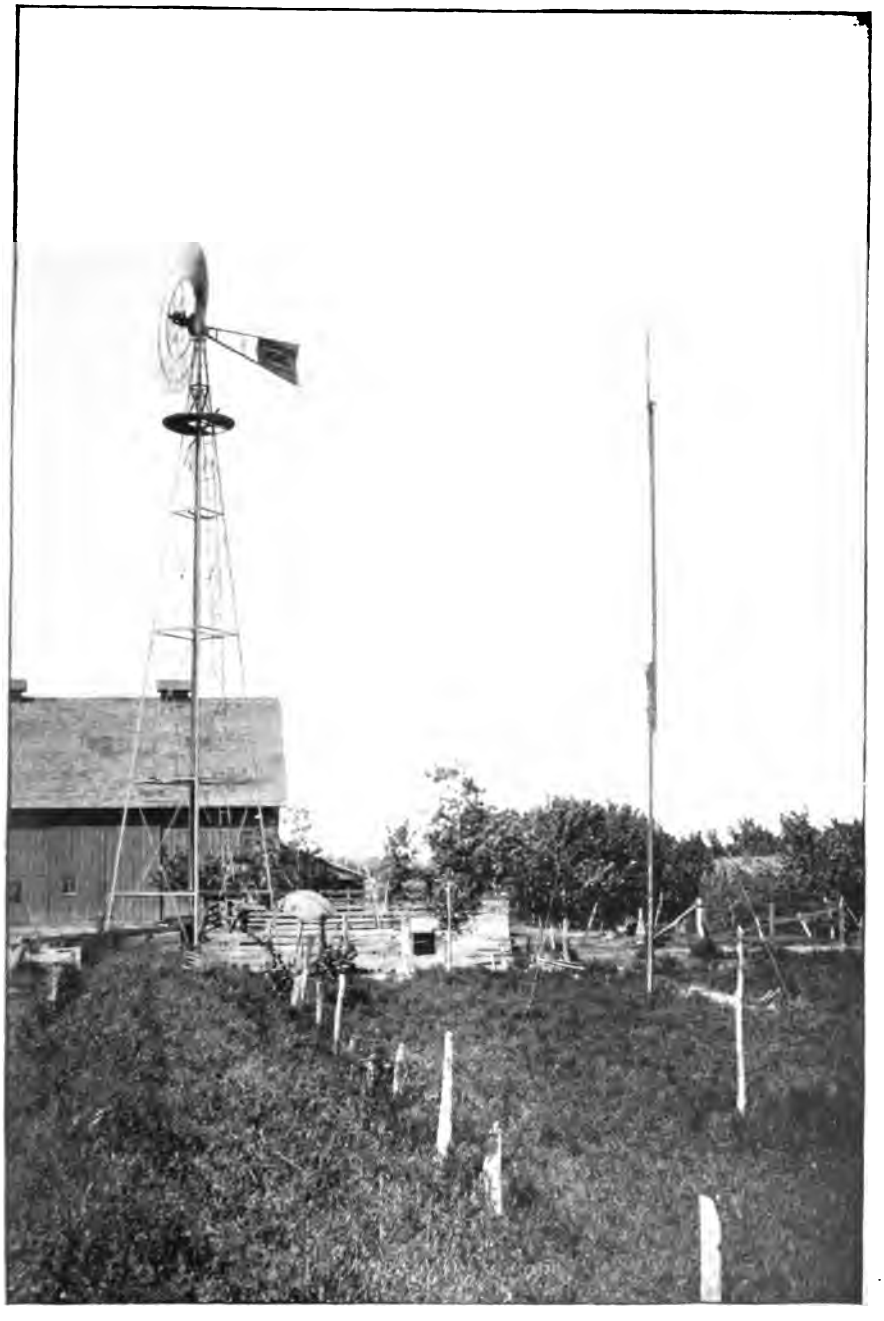


FIG. 6.—Diagram showing results with mill No. 2—12-foot Woodmanse Mogul.



VIEW OF MILL NO. 2 (12-FOOT WOODMANSE MOGUL) AND ANEMOMETER.

miles an hour. It ascends very rapidly, reaching a maximum at 18 miles an hour, and giving 60 strokes to the mile. The rest of the curve to 30 miles has a gentle slope. The number of strokes per minute increases from about 5 at 12 miles to about 25 at 30 miles an hour, and will continue to increase to probably 28 a minute in a 40-mile wind.

Mill No. 3.—This is a 12-foot Aermotor manufactured by the Aermotor Company, of Chicago, Illinois. Pl. IV shows the mill with its tower, pump, and pond, and fig. 7 shows its working parts. This mill had been in use about one year at the time of test, and all of the parts were in good working order. The tower is of wood, the axis of the wheel being 30 feet above the ground. The exposure is very good. The wheel has 18 curved sails, each 44 by 18¾ by 7¾ inches, set at an angle of 31° to the plane of the wheel. It is back-geared, 3⅓ to 1, and is held in the wind by a spring. The pump is of the Stone type, shown in figs. 8 and 9; the check valve is of the single-flap variety, the plunger valve of the double-flap variety. The cylinder is 9½ inches in diameter, the supply pipe 4 inches in diameter, and the discharge pipe 10 inches in outside diameter; the length of stroke is 12 inches and the discharge per stroke 14½ quarts. The well is 4 feet by 4 feet to a depth of 8 feet—nearly down to water. From that point to a depth of 18 feet it is 3 feet in diameter; and from there three pipes, 12 inches in diameter, extend down 5 feet farther. The lift at the time of test was 13¾ feet, the barometric pressure 27.2 inches, and the temperature 85° F. The water is pumped into a pond

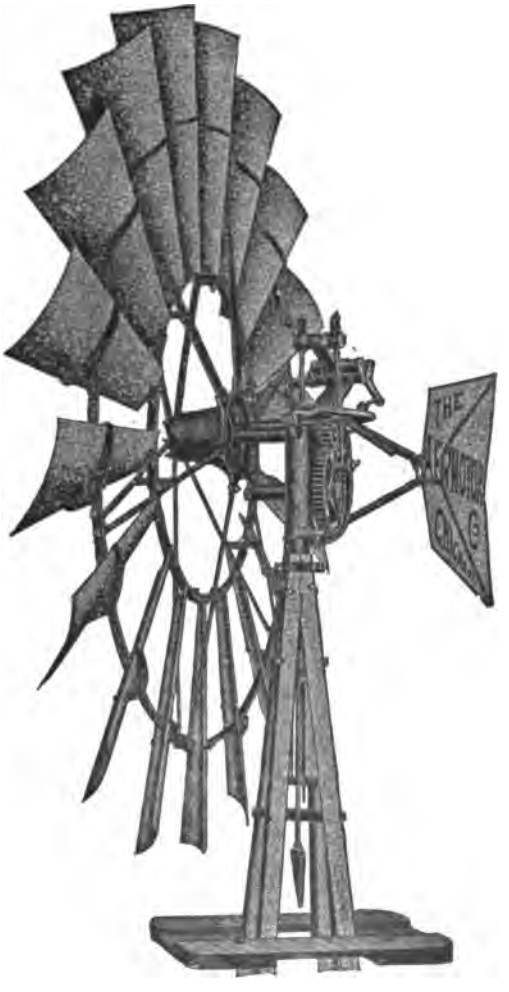


FIG. 7.—Working parts of Aermotor

80 feet by 75 feet, and a depth of 22 inches can be drawn off. The cost of the plant, including mill, tower, pump, and pond, was $145. The greatest number of strokes per minute is probably 27 or 28 in a 40-mile wind. In general appearance this curve is seen to resemble that of mill No. 2, but it is about 4 miles farther to the left, due to lighter load. The results of the tests are as follows:

FIG. 8.—Stone pump.

Results of tests of mill No. 3—12-foot Aermotor.

[Load per stroke, 415.3 foot-pounds.]

Wind velocity per hour.	Revolutions of wind wheel per minute.	Strokes of pump per minute.	Gallons pumped per minute.	Useful horse-power.
8 miles	17.7	5.3	19.3	0.067
12 miles	40.0	12.0	43.5	0.151
16 miles	54.7	16.4	59.5	0.207
20 miles	66.0	19.8	71.8	0.250
25 miles	77.0	23.1	83.6	0.291
30 miles	83.3	25.0	90.6	0.315

The curve shown in fig. 10 is for a rather lightly loaded mill (415.3 foot-pounds per stroke). It starts at a velocity of 6 to 7 miles an hour, ascends less rapidly than the one shown in fig. 6, attains a maximum at about 15 miles an hour, when the number of strokes per mile is 62, and then descends slowly, reaching 50 strokes at 30 miles. The number of strokes per minute increases from about 5 at 8 miles to 25 at 30 miles.

Mill No. 4.—This mill, shown in the foreground of Pl. V, is an 8-foot Ideal windmill manufactured by the Stover Manufacturing Company, of Freeport, Illinois. It had been in use about one year, and all of the parts were in good condition. The tower is of wood, the axis of the wheel being 48 feet above the ground.

The wheel has 15 sails, each 16⅛ by 7 by 30 inches, set at an angle of 29° with the plane of the wheel. It is back-geared, 2½ to 1, and is held in the wind by a spring. The pump is of the Stone make. The diameter of the discharge pipe is 5⅝ inches, of the supply pipe 3 inches. The length of stroke is 8 inches. The plunger and check valves are of the single-flap variety. The well is 2¾ feet by 2¾ feet down nearly to water—a depth of 5¼ feet. The 3-inch supply pipe extends down to a depth of 14 feet, and on the end of it is a 3-inch well point 6 feet long. The lift may vary from 8½ to 20 feet. It was probably about 12 feet at the time of tests. The discharge per stroke was 2 quarts. The mean barometric pressure was 27.19 inches, the mean tempera-

VIEW OF MILL NO. 3—12-FOOT AERMOTOR.

ture 83° F. The water is pumped into a pond 115 feet by 31 feet and 3 feet deep. The cost of the plant, including mill, pump, well, and pond, was $80. The results of the tests are as follows:

Results of tests of mill No. 4—8-foot Ideal.

[Load per stroke, 50 foot-pounds.]

Wind velocity per hour.	Revolutions of wind wheel per minute.	Strokes of pump per minute.	Gallons pumped per minute.	Useful horse-power.
12 miles	25.5	10.2	5.1	0.015
16 miles	48.2	19.3	9.6	0.029
20 miles	63.2	25.3	12.6	0.038
25 miles	70.3	28.1	14.1	0.043
30 miles	62.5	25.0	12.5	0.038

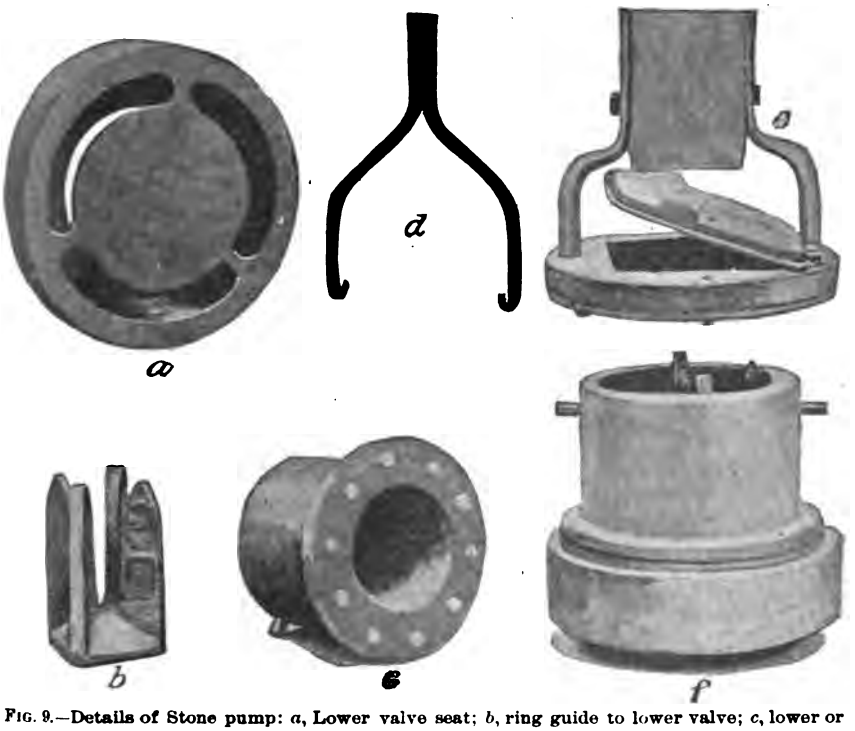


FIG. 9.—Details of Stone pump: *a*, Lower valve seat; *b*, ring guide to lower valve; *c*, lower or check valve; *d*, hook for removing lower valve; *e*, plunger and valve; *f* is *a*, *b*, and *c* combined.

The curve shown in fig. 11, although for a rather lightly loaded mill—50 foot-pounds per stroke—shows that the mill starts in a 10-mile to an 11-mile wind. The maximum is reached at 19 miles, with a speed of 78 strokes. The right side of the curve is quite steep, a characteristic of this make of mill. Mill No. 18 is the same size and make as this mill, and yet with a load of 89.2 foot-pounds it starts in a 7-mile to an 8-mile wind, reaching a maximum at about 13 miles, at

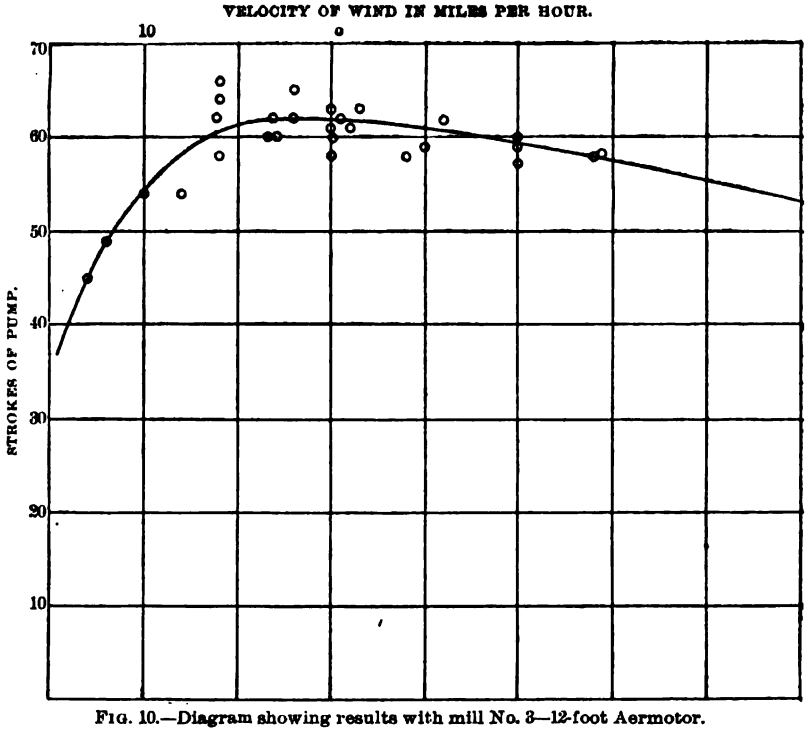


FIG. 10.—Diagram showing results with mill No. 3—12-foot Aermotor.

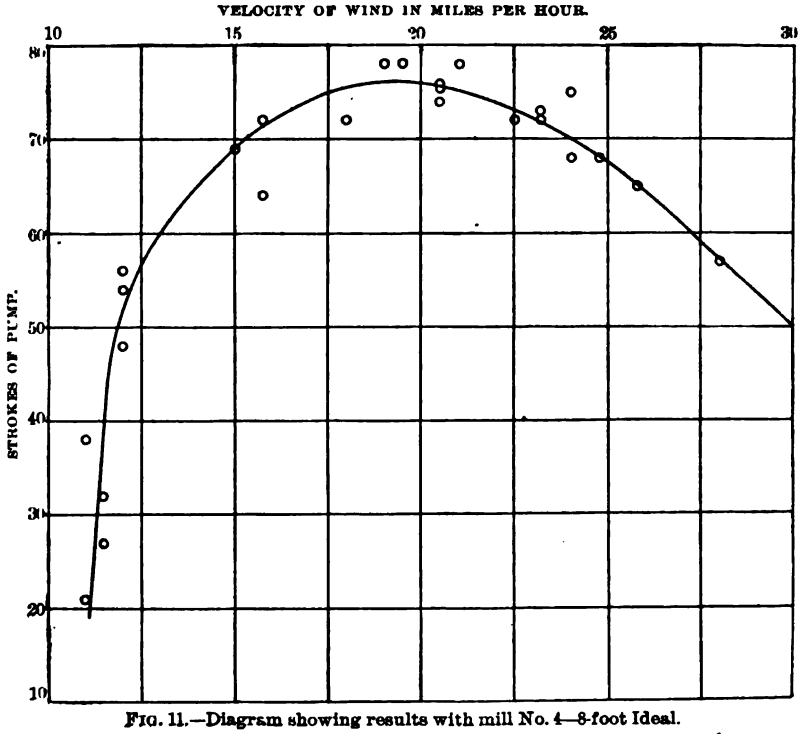


FIG. 11.—Diagram showing results with mill No. 4—8-foot Ideal.

VIEW OF MILLS NO. 4 (8-FOOT IDEAL) AND NO. 5 (8-FOOT AERMOTOR).

a speed of 104 strokes per mile. A second test, when the spring that holds the wind wheel in the wind was tightened somewhat, gave the maximum at a velocity of about 15 miles, with a pump speed of about 114 strokes. The difference appeared to be due to the difference in pumps and wells. The rapid fall in the curve to the right of the highest point is due to the easy governing of the mill.

Mill No. 5.—This mill, shown in the background of Pl. V, is an 8-foot Aermotor manufactured by the Aermotor Company, of Chicago, Illinois. The tower is of wood, and is 28.5 feet high to the axis of wheel. The exposure is good, and all of the parts were in good working order at the time of tests, the plant having been in use about one year. The wheel has 18 curved sails, each 30 by 12½ by 5½ inches, making an angle of 29½° with the plane of the wheel. It is back-geared, 3⅓ to 1. The pump is of the Stone make. The discharge pipe is 6 inches in diameter, the supply pipe 3 inches in diameter. The valves (check and plunger) are of the single-flap variety. The length of stroke is 8 inches. The well is 4 feet by 4 feet to water, a depth of 10.5 feet. A 12-inch wooden curb extends 12 feet farther into the sand and gravel. The discharge per stroke was 3½ quarts, and the lift 13 feet. The cost of plant, including pond, was $80.

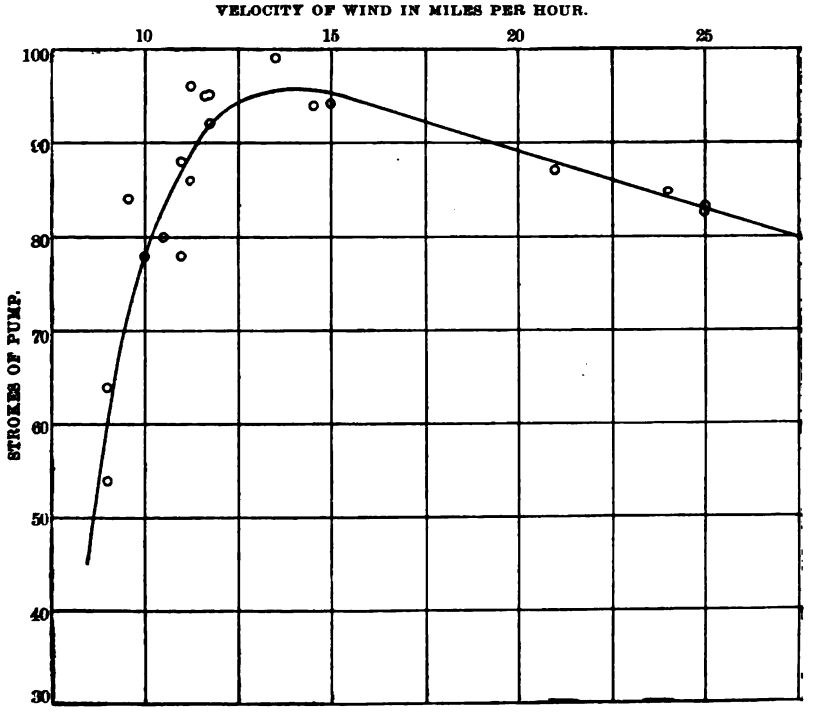


FIG. 12.—Diagram showing results with mill No. 5—8-foot Aermotor.

The results of the tests of mill No. 5 are as follows:

Results of tests of mill No. 5—8-foot Aermotor.

[Load per stroke, 94.9 foot-pounds.]

Wind velocity per hour.	Revolutions of wind wheel per minute.	Strokes of pump per minute.	Gallons pumped per minute.	Useful horse-power.
12 miles	62.0	18.6	16.3	0.053
16 miles	83.7	25.1	22.0	0.072
20 miles	99.3	29.8	26.1	0.086
25 miles	115.3	34.6	30.3	0.099
30 miles	128.3	38.5	33.7	0.111

This 8-foot mill, with a load of 95 foot-pounds, is seen to start in an 8-mile to a 9-mile wind (fig. 12), reaching a maximum at 13 to 15 miles, with 95 strokes per mile. At 30 miles an hour it is making 77 strokes per mile, or 39 strokes per minute. This curve indicates a rather heavily loaded mill.

Mill No. 6.—This mill, shown in the background of Pl. IV, is an 8-foot Gem, manufactured by the United States Wind Engine and Pump Company, of Kansas City, Missouri. The working parts of the mill are shown in Pl. VI. The exposure was good and all of the parts were in good working order, the mill having been in use only about one year at the time of tests. The wheel has 24 curved sails, each 30½ by 10 by 4½ inches, set at an angle of 35° with the plane of the wheel. It is back-geared, 3 to 1. The wheel is held in the wind by means of a weight. The pump is of the Stone make. The discharge pipe is 6 inches in diameter, the supply pipe 4 inches in diameter. The length of stroke is 8 inches. The well is open to the water—a depth of 6½ feet. The supply pipe is on a well point, the end of which is 16 feet below the surface of the ground. The lift was $9\frac{7}{12}$ feet and the discharge per stroke 3.9 quarts. The tower is of wood, and is 24 feet high to the axis of the wheel. The mean barometric pressure was 27.02 inches, the mean temperature 85° F. The plunger valve is of the double-flap variety, and the check valve of the single-flap variety. The results of the tests are as follows:

Results of tests of mill No. 6—8-foot Gem.

[Load per stroke, 77.6 foot-pounds.]

Wind velocity per hour.	Revolutions of wind wheel per minute.	Strokes of pump per minute.	Gallons pumped per minute.	Useful horse-power.
12 miles	37.2	12.4	12.1	0.029
16 miles	53.7	17.9	17.5	0.042
20 miles	66.0	22.0	21.5	0.051
25 miles	76.2	25.4	24.7	0.059
30 miles	85.5	28.5	27.8	0.065

Comparing the number of strokes per minute of mills Nos. 4, 5, and 6, it is seen that although No. 5 is carrying a much heavier load than either of the other mills, it makes more strokes and does much more work at all velocities.

Mill No. 7.—This is a 12-foot Aermotor similar to mill No. 3. The tower is of steel, having a height of 31 feet to the axis of the wheel. The exposure was good and all of the parts were in good working order, the plant having been in use less than one year when tests were made. The pump is of the Stone make and is like that of mill No. 3, except that the check valve is of the solid-lift variety. The lift was 15½ feet and the discharge 14.3 quarts per stroke. The water is pumped into a pond 135 feet by 50 feet by 2½ feet.

Comparing the results of the tests of this mill with those of mill No. 3, it is seen that the latter is somewhat more heavily loaded than the former and makes a few less strokes per minute, but that its horsepower is a little greater. The effect of the larger load is shown.

The results of the tests of mill No. 7 are as follows:

Results of tests of mill No. 7—12-foot Aermotor.

[Load per stroke, 461.9 foot-pounds.]

Wind velocity per hour.	Revolutions of wind wheel per minute.	Strokes of pump per minute.	Gallons pumped per minute.	Useful horse-power.
12 miles	38.0	11.4	40.7	0.160
16 miles	52.7	15.8	56.5	0.221
20 miles	63.3	19.0	67.9	0.266
25 miles	73.7	22.1	79.0	0.309
30 miles	78.3	23.5	84.0	0.329

Mill No. 8.—This is a 10-foot Star wooden mill, manufactured by Bradley, Wheeler & Company, of Kansas City, Missouri. The tower is of wood, and the axis of the wheel 35½ feet above the ground. The water is pumped into an elevated tank 20 feet above the surface of the ground and is used for irrigation. The wheel has 60 plane sails, each 37 by 5 by 2¾ inches, set at an angle of 33° to the plane of the wheel. It is held in the wind by means of a weight. It is not back-geared, a stroke of the pump being made to each revolution of the wheel. The supply pipe is 2 inches in diameter and terminates in a well point, the end of which is 18 feet below the surface of the ground. The discharge pipe is 1¼ inches in diameter. The cylinder is 3 inches in diameter, the length of stroke 5 inches. The lift may vary between 28½ and 37 feet. It was estimated to be about 30 feet at the time of measurement. The discharge per stroke was 0.24 quart. The cylinder leaked some at the time of tests. After a new cylinder was put in the discharge per stroke was increased to 0.40 quart. The mean

barometric pressure was 27.04 inches, the mean temperature 78° F. The results of the tests are as follows:

Results of tests of mill No. 8—10-foot Star wooden mill.

[Load per stroke, 15 foot-pounds.]

Wind velocity per hour.	Revolutions of wind wheel per minute.	Strokes of pump per minute.	Gallons pumped per minute.	Useful horse-power.
8 miles	28.0	28.0	1.7	0.013
12 miles	30.0	30.0	1.8	0.014

These results show the effect of the very light load and the readiness with which the wind wheel turns out of the wind. It makes 28 strokes per minute in an 8-mile wind and less than that in a 16-mile or higher wind.

Mill No. 9.—This is a 16-foot Aermotor. The tower is of steel, and the axis of the wheel is 30 feet above the ground. The wheel has 18

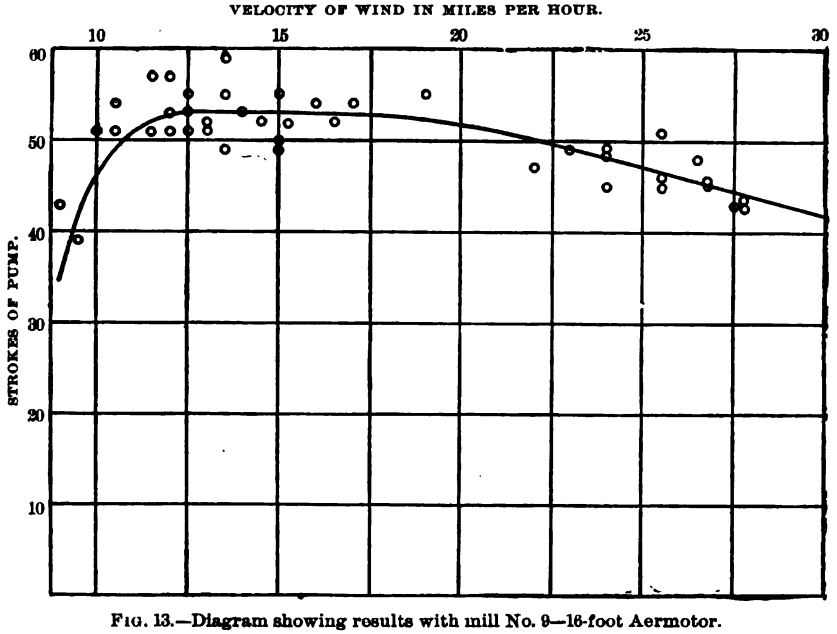


FIG. 13.—Diagram showing results with mill No. 9—16-foot Aermotor.

curved sails, each 59 by 25¾ by 10½ inches, set at an angle of 30° with the plane of the wheel. It is back-geared, 3 to 1. The discharge pipe is 12 inches in diameter, the supply pipe 6 inches in diameter, the cylinder 8 inches in diameter. The stroke is 16 inches. The well is 4 feet by 6 feet to a depth of 23 feet, 2 feet by 2 feet for the next 8 feet, and 18 inches in diameter for the next 14 feet. The water was 39½ feet below the surface of the ground. The lift was 44¼ feet and the

discharge per stroke 11 quarts. The check valve is of the single-flap variety and the plunger valve of the double-flap variety. The mean barometric pressure was 27.04 inches, and the mean temperature 93° F. This plant had been in use about three years. The results of the tests are as follows:

Results of tests of mill No. 9—16-foot Aermotor.

[Load per stroke, 1,013 foot-pounds.]

Wind velocity per hour.	Revolutions of wind wheel per minute.	Strokes of pump per minute.	Gallons pumped per minute.	Useful horse-power.
12 miles	31.8	10.6	29.1	0.325
16 miles	42.3	14.1	38.8	0.433
20 miles	51.6	17.2	47.3	0.548
25 miles	58.8	19.6	53.9	0.601
30 miles	63.0	21.0	57.7	0.644

The curve shown in fig. 13 starts at a wind velocity of 8 to 9 miles, and reaches a maximum at 13 miles, with a speed of 53 strokes per mile. From that point to a velocity of about 19 miles the curve is nearly horizontal; after 19 miles it descends slowly to 32 miles, with 38 strokes per mile. The speed increases from 11 strokes per minute at 12 miles to 21 strokes per minute at 30 miles an hour.

Mill No. 10.—This is an 8-foot Ideal. The tower is of wood, the axis of the wheel being 30 feet above the ground. The wheel has 15 curved sails, each 31 by 19 by 7 inches, set at an angle of 29½° with the plane of the wheel. It is back-geared, 2½ to 1. The supply pipe is 1½ inches in diameter, the cylinder 2½ inches in diameter. The pump is a common hand pump, with lift valve of the flap form and plunger of the lift variety. The valves leak some, as the discharge is greater when the pump is working rapidly than when it is working slowly. The supply pipe is on a well point 2 feet long and 1½ inches in diameter, the lower end of which is 50 feet below the surface of the ground. The lift was 33 feet and the discharge per stroke one-third of a quart when pumping quite rapidly. The mean barometric pressure was 26.94 inches, the mean temperature 97° F. The results of the tests are as follows:

Results of tests of mill No. 10—8-foot Ideal.

[Load per stroke, 22.8 foot-pounds.]

Wind velocity per hour.	Revolutions of wind wheel per minute.	Strokes of pump per minute.	Gallons pumped per minute.	Useful horse-power.
8 miles	35.2	14.1	1.2	0.010
12 miles	62.5	25.0	2.1	0.017
16 miles	83.2	33.3	2.8	0.023
20 miles				0.032

Mill No. 11.—This is a 12-foot Ideal, the working parts of which are shown in fig. 14. The tower is of steel, the axis of the wheel being 30 feet above the ground. The exposure was good, and all of the

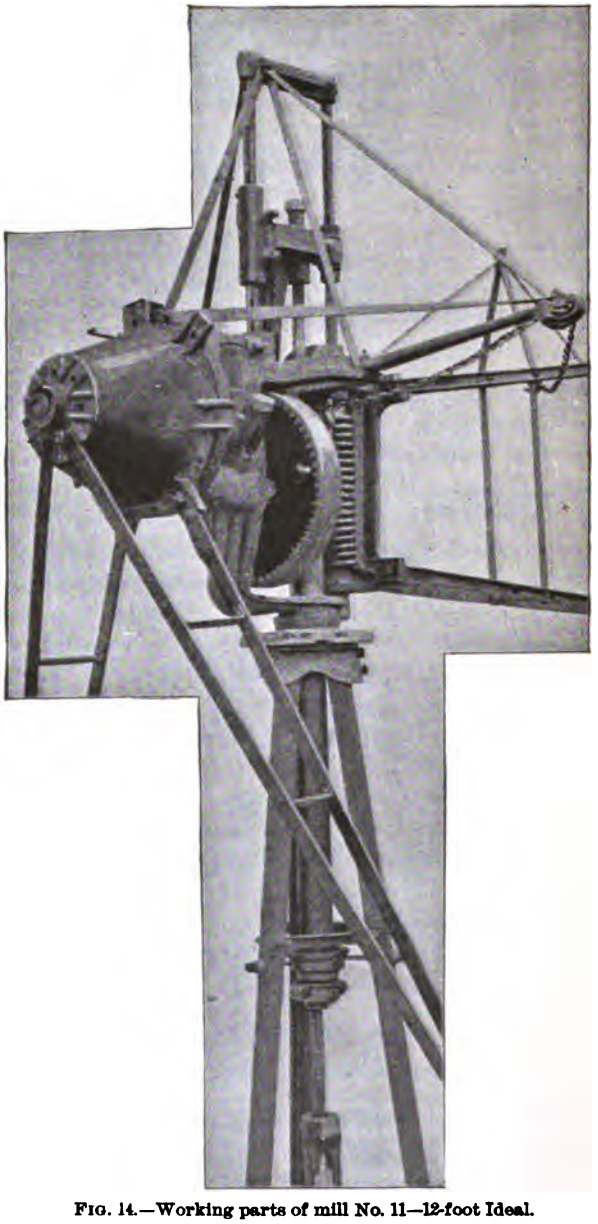


FIG. 14.—Working parts of mill No. 11—12-foot Ideal.

parts were in good working order when mill was tested. The wheel has 21 curved sails, each 31 by 19 by 7 inches, set at an angle of 29½° to the plane of the wheel. It is back-geared, 2½ to 1, and the wheel

is held in the wind by a spring. The discharge pipe is 8 inches in diameter, and the length of stroke is 12 inches. The supply pipe consists of two 3-inch pipes 14 feet long, each terminating in a 3-inch well point 3 feet long. The valves (check and plunger) are of the single-flap variety. The water was 39 feet below the surface of the ground. The lift, as nearly as could be ascertained at the time of measurement, was 45 feet, the discharge per stroke 9 quarts. The water is pumped into a pond 60 feet by 40 feet by 6 feet. This plant had been in use about three years. The mean barometric pressure was 26.91 inches, the mean temperature 91° F. The results of the tests are as follows:

Results of tests of mill No. 11—12-foot Ideal.

[Load per stroke, 843.7 foot-pounds.]

Wind velocity per hour.	Revolutions of wind wheel per minute.	Strokes of pump per minute.	Gallons pumped per minute.	Useful horse-power.
12 miles	12.0	4.8	10.8	0.123
16 miles	31.7	12.7	28.6	0.325
20 miles	47.0	18.8	42.3	0.481
25 miles	58.2	23.3	52.5	0.600
30 miles	62.5	25.0	56.2	0.639

Mills Nos. 9 and 11 pump water into the same pond from the same depth. It will be seen from these results that for wind velocities of

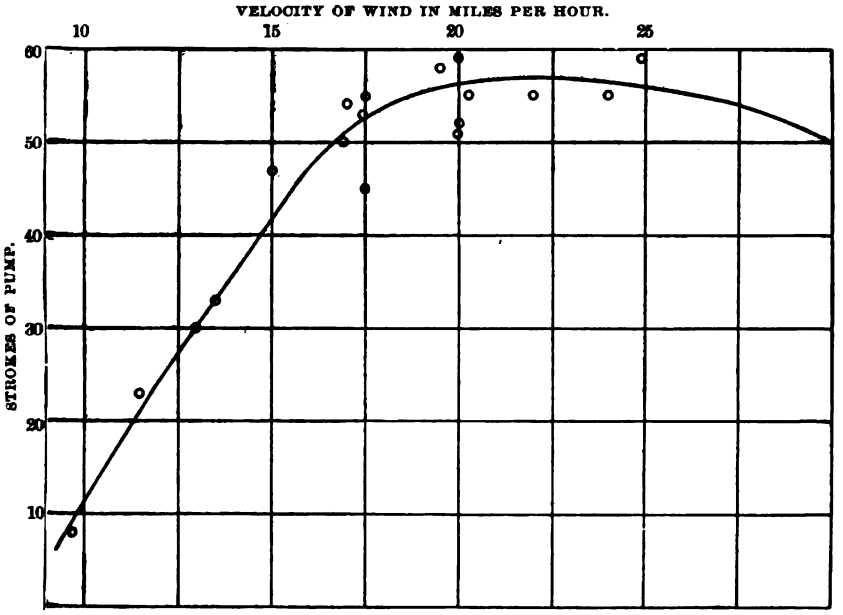


FIG. 15.—Diagram showing results with mill No. 11—12-foot Ideal.

20 miles or more the 12-foot mill is pumping nearly as much water as the 16-foot mill; for velocities of 12 miles or less the 16-foot mill is pumping much more water than the 12-foot mill.

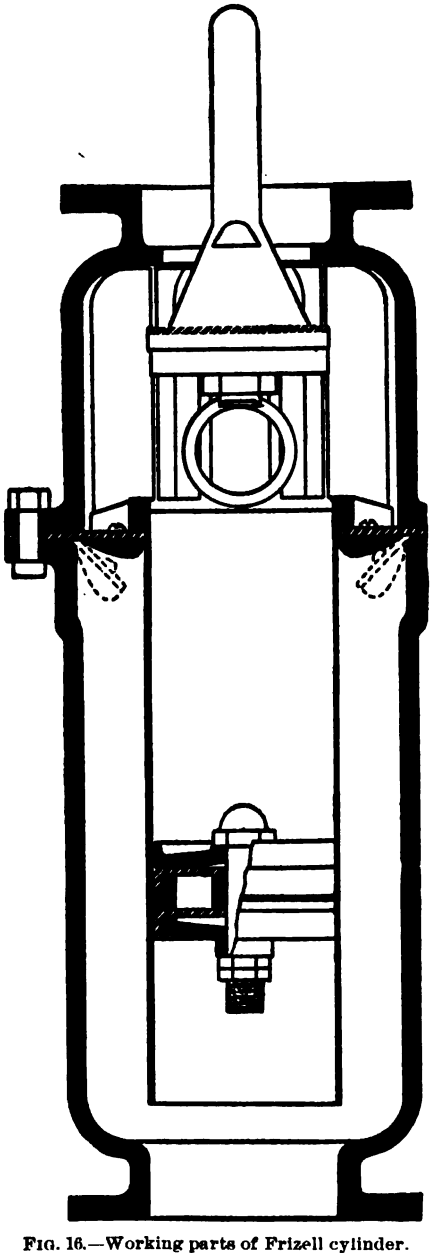
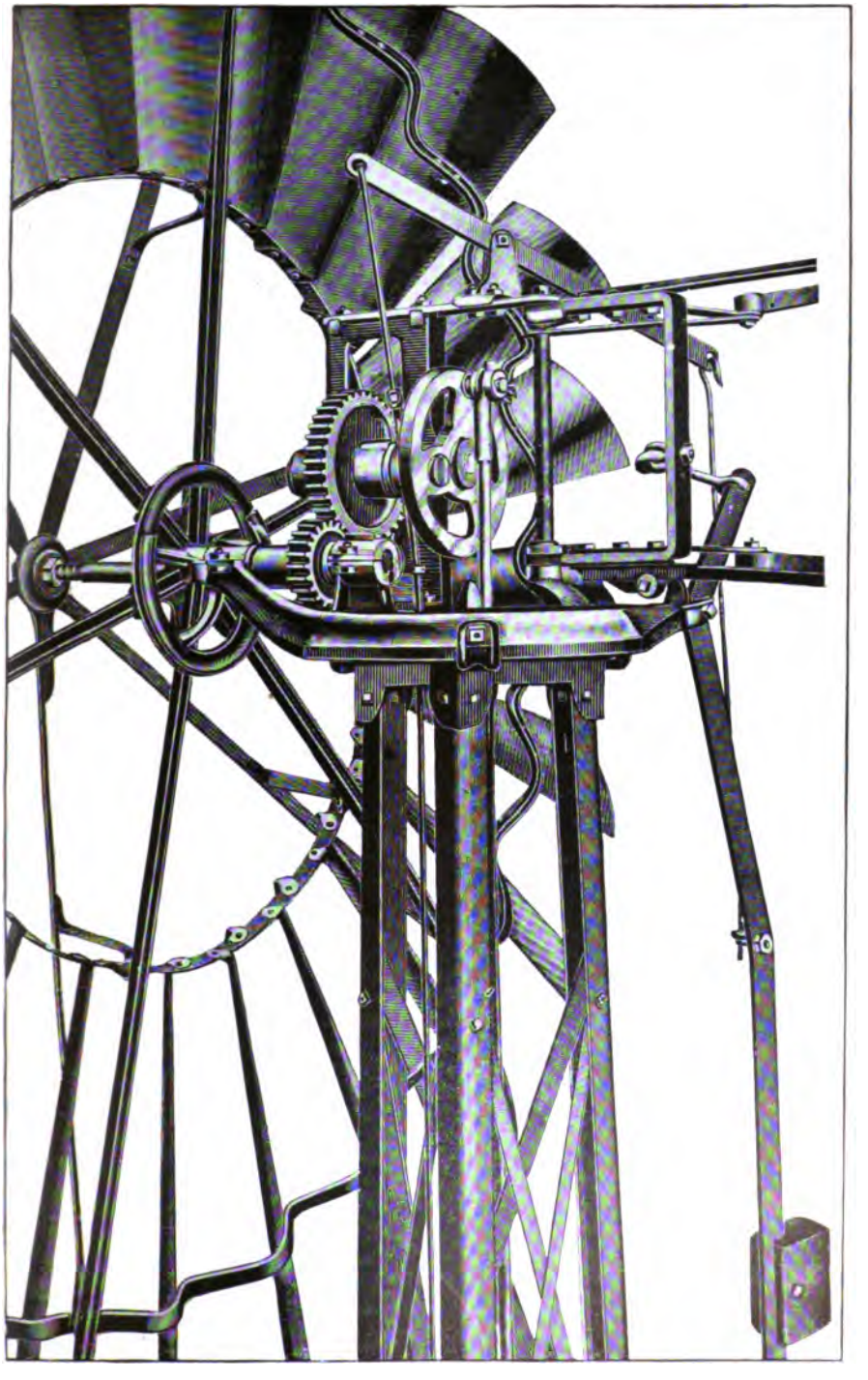


FIG. 16.—Working parts of Frizell cylinder.

This curve (fig. 15) is for a heavily loaded (843.7 foot-pounds per stroke) 12-foot mill. It starts with a velocity of about 10 miles an hour, and reaches a maximum at about 23 miles, with a speed of 57 strokes per mile. At 30 miles it is making 51 strokes per mile. The maximum point of this curve is much farther to the right than that of any other curve. The number of strokes increases from 5 per minute at 12 miles to 25 per minute at 30 miles.

Mill No. 12.—This is a 14-foot Ideal, shown in Pl. VII. The tower is of steel and is 30 feet high to the axis of the wheel. The wheel has 24 curved sails, each 48¾ by 17½ by 8 inches, set at an angle of 30° with the plane of the wheel. It is back-geared, 2½ to 1. The pump is of the Frizell make (shown in fig. 16). The discharge pipe is 10 inches in diameter, the cylinder 9½ inches in diameter, the supply pipe 6 inches in diameter, terminating in a well point 10 feet long and 6 inches in diameter, the lower end of which is 32 feet below the surface of the ground. The lift, as nearly as could be estimated, was 11 feet, the discharge per stroke 11.6 quarts. The mean barometric pressure was 27.04 inches, the mean temperature 81° F. The water is pumped into a reservoir 100 feet by 100 feet by 3 feet deep. The pump had been in use about one year. The results of the tests are as follows:



WORKING PARTS OF MILL NO. 6—8-FOOT GEM.

Results of tests of mill No. 12—14-foot Ideal.

[Load per stroke, 263.5 foot-pounds.]

Wind velocity per hour.	Revolutions of wind wheel per minute.	Strokes of pump per minute.	Gallons pumped per minute.	Useful horse-power.
8 miles	7.7	3.1	8.9	0.025
12 miles	27.2	10.9	30.1	0.087
16 miles	39.3	15.7	45.1	0.125
20 miles	48.0	19.2	55.2	0.153
25 miles	53.7	21.5	61.8	0.173

This curve (fig. 17) is for a very lightly loaded (263.5 foot-pounds) 14-foot mill. This load is only 31 per cent of that of the 12-foot mill, No. 11. The curve starts in a 7-mile to an 8-mile wind, and reaches a maximum at 15 miles, with a speed of 58 strokes per mile of wind. At 30 miles the speed is 47 strokes. Although this is a very lightly

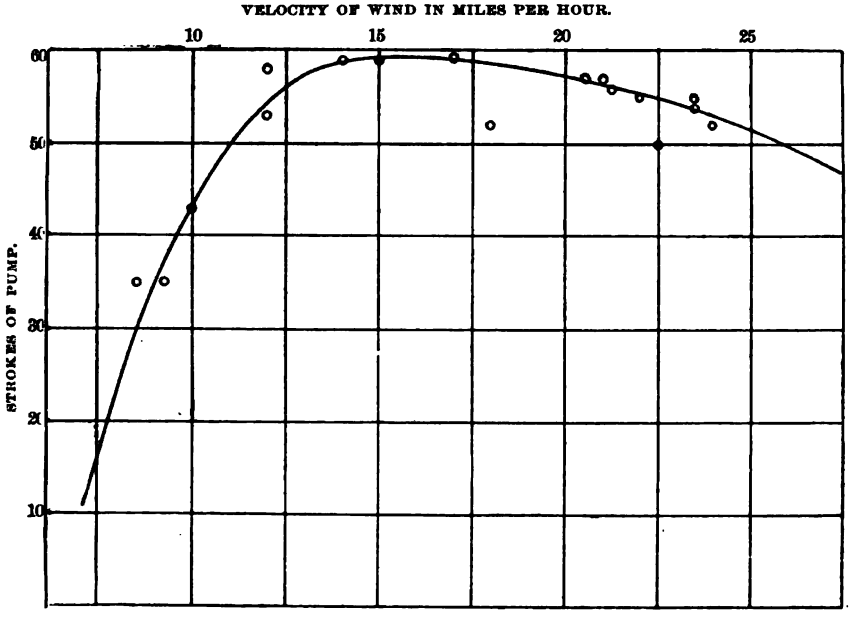


FIG. 17.—Diagram showing results with mill No. 12—14-foot Ideal.

loaded mill it does not make many strokes per minute. In fact, although it is not as heavily loaded as the 12-foot mill, No. 3, it does not make as many strokes per minute as the latter mill, and it is producing much less power than mill No. 3.

Mill No. 13.—This is a 12-foot Aermotor (shown in Pl. VIII). The tower is of wood, with the axis of the wheel 25 feet above the ground. The exposure was good and the plant in excellent condition, having

been in use about one year at the time of tests. The wheel is the same as that of No. 3. The pump is of the Stone make. The discharge pipe is 10 inches in diameter, the supply pipe 5 inches in diameter, on a well point 10 feet long, the lower end of which is 17 feet below the surface of the ground. The length of stroke is 12 inches. The plunger valve is of the double-flap type, and the check valve of the single-flap type. The discharge per stroke at the time of test was 14.4 quarts and the lift about 11 feet. The mean barometric pressure was 27.09 inches, the mean temperature 91° F. The results of the tests are as follows:

Results of tests of mill No. 13—12-foot Aermotor.

[Load per stroke, 330 foot-pounds.]

Wind velocity per hour.	Revolutions of wind wheel per minute.	Strokes of pump per minute.	Gallons pumped per minute.	Useful horse-power.
12 miles	36.7	11.0	39.6	0.110
16 miles	57.0	17.1	61.6	0.171
20 miles	70.0	21.0	75.6	0.210
25 miles	82.0	24.6	88.6	0.247
30 miles	91.7	27.5	99.0	0.275

The important difference between this plant and No. 3 is that the latter has a 4-inch supply pipe and an open well, while the former has a 5-inch supply pipe on a well point. The useful load per stroke of mill No. 13 is 20 per cent less than that of mill No. 3, and the number of strokes per minute of No. 13 is slightly greater than that of No. 3. It appears that the well point offers some resistance, but how much can not be said from this data.

Mill No. 14.—This is a 12-foot Gem, like the one shown in Pl. IX, on a 60-foot steel tower. The pump is of the Gause make. The cylinder is 8 inches in diameter, the length of stroke 9 inches. The supply is from a 12-inch pipe in an open well. The discharge per stroke was 9¾ quarts and the lift 15½ feet. The wind velocity was not measured.

Mill No. 15.—This is a 10-foot Gem similar to that shown in Pl. IX. The tower is of wood, the axis of the wheel being 34 feet above the ground. The mill was in good working order, but the exposure was not good, on account of trees. The wheel has 24 sails, each 36 by 11 by 4¾ inches, set at an angle of 35° with the plane of the wheel. It is back-geared, 3 to 1. The pump is of the Stone make. The discharge pipe is 8 inches in diameter. The supply pipe is on a 3-inch well point 8 feet long, the lower end of which is 21½ feet below the surface of the ground. The plunger valve is of the single-flap form. Depth to water is 10 feet. The discharge per stroke was 7 quarts, the lift about 15 feet. The mean barometric pressure was 27.05 inches, the mean temperature 84° F.

VIEW OF MILL NO. 12—14-FOOT IDEAL.

VIEW OF MILL NO. 13—12-FOOT AERMOTOR.

The results of the tests are as follows:

Results of tests of mill No. 15—10-foot Gem.

[Load per stroke, 219 foot-pounds.]

Wind velocity per hour.	Revolutions of wind wheel per minute.	Strokes of pump per minute.	Gallons pumped per minute.	Useful horse-power.
12 miles	23.4	7.8	13.6	0.053
16 miles	35.7	11.9	21.0	0.082
20 miles	44.1	14.7	25.7	0.101
25 miles	43.8	14.6	25.5	0.099

This mill revolves very slowly, indicating a heavy load. Its useful horsepower, however, is little greater than that of mill No. 5.

Mill No. 16.—This is a 10-foot Halliday, pumping water into the same pond as No. 15. It is similar to the mill shown in fig. 20. The tower is of wood, the axis of the wheel being 28 feet above the ground. The wheel has 78 sails, each 36¼ by 4 by 2¼ inches, set at an angle of 35.5° to the plane of the wheel. It is not back-geared. The pump is of the Gause make, with a discharge pipe 6 inches in diameter and a supply pipe 4 inches in diameter. There is a 6-inch galvanized-iron pipe, forming an open well, extending 15 feet into the water. The depth to water was 11 feet, the lift 16 feet, and the discharge per stroke 3 quarts. The mean barometeric pressure was 27.02 inches, the mean temperature 94° F. The results of tests are as follows:

Results of tests of mill No. 16—10-foot wooden Halliday.

[Load per stroke, 100 foot-pounds.]

Wind velocity per hour.	Revolutions of wind wheel per minute.	Strokes of pump per minute.	Gallons pumped per minute.	Useful horse-power.
8 miles	4.0	4.0	3.0	0.012
12 miles	22.6	22.6	16.9	0.067
16 miles	33.9	33.9	25.4	0.103
20 miles	42.7	42.7	32.0	0.130
25 miles	52.5	52.5	39.3	0.159

This 10-foot wooden mill is seen to be doing more useful work than the 10-foot steel Gem, No. 15. Usually, however, the direct-stroke wooden mills do less work than the back-geared steel mills of the same size.

Mill No. 17.—This is a 12-foot improved Gem on a 30-foot steel tower. The wheel has 32 curved sails, each 42 by 11½ by 4¾ inches, set at an angle of 37° with the plane of the wheel. It is back-geared, 2 to 1. The pump is of the Gause type, with an 8-inch discharge pipe, a 4-inch supply pipe, 12 inches stroke, and an open well formed of a 12-inch wooden casing. The depth to water was 17¼ feet, and the

discharge per stroke 8¾ quarts. The mean barometric pressure was 27.05 inches, the mean temperature 93° F. The results of the tests are as follows:

Results of tests of mill No. 17—12-foot improved Gem.

[Load per stroke, 385 foot-pounds.]

Wind velocity per hour.	Revolutions of wind wheel per minute.	Strokes of pump per minute.	Gallons pumped per minute.	Useful horse-power.
12 miles	12.0	6.0	12.7	0.070
16 miles	25.6	12.8	27.2	0.149
20 miles	34.6	17.3	36.8	0.202

This mill, although nearly new, does not work well. It is out of plumb. Only a few measurements of the number of strokes per mile of wind were made.

Mill No. 18.—This is an 8-foot Ideal on a 36-foot wooden tower. The exposure was good and the parts in good working order. The wheel is like that of mill No. 4. The pump is of the Stone make, with a 6-inch discharge pipe. There is no supply pipe, the cylinder being under water, with 3 inches opening to it from below. The check valve is of the lift variety, the plunger valve of the single-flap variety. The well is dug to a depth of 8 feet. It is 4⅓ feet in diameter. In the bottom a 10-inch galvanized-iron pipe extends down several feet. It was 11 feet to water. The lift was 14⅓ feet and the discharge per stroke 2.92 quarts. The mean barometric pressure was 27.01 inches, the mean temperature 83° F. The cost of the plant, including pond, was $125. The results of the tests are as follows:

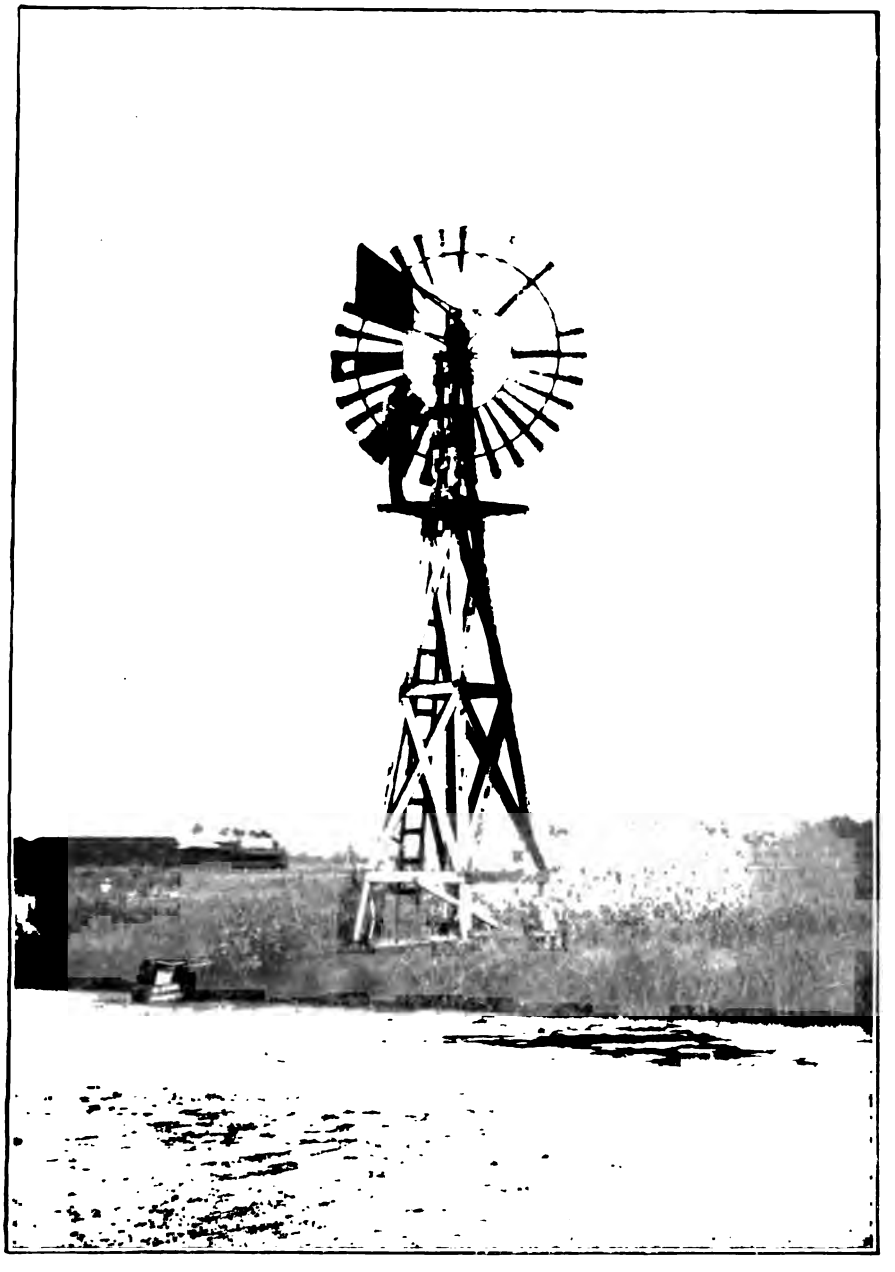
Results of tests of mill No. 18—8-foot Ideal.

[Load per stroke, 89.2 foot-pounds.]

Wind velocity per hour.	Revolutions of wind wheel per minute.	Strokes of pump per minute.	Gallons pumped per minute.	Useful horse-power.
8 miles	20.0	8.0	5.8	0.022
12 miles	50.5	20.2	14.6	0.054
16 miles	65.2	26.1	18.9	0.070
20 miles	70.0	28.0	20.3	0.076
25 miles	68.7	27.5	19.9	0.074

After the spring which holds the wind wheel of this mill in the wind was tightened, the number of strokes per mile of wind was increased from 98 to 114 in a 16-mile wind, from 84 to 102 in a 20-mile wind, and from 66 to 79 in a 25-mile wind.

Mill No. 19.—This is a 12-foot Gem (shown in Pl. IX) on a 30-foot wooden tower. The exposure was good and the mill in good working order. The wheel is like that of mill No. 17. The pump is of the Stone make, with a 10-inch discharge pipe and a 4-inch supply



VIEW OF MILL NO. 19—12-FOOT GEM.

pipe. The length of the stroke is 10 inches. The supply pipe is on a 4-inch well point 9 feet long, the end of which is 23 feet below the surface of the ground. The check valve is of the lift type and the plunger valve of the single-flap type. The lift was about 18 feet and the discharge per stroke 12 quarts. The mean barometric pressure was 27.13 inches, the mean temperature 70° F. The water is pumped into a reservoir 120 feet by 60 feet. The results of the tests are as follows:

Results of tests of mill No. 19—12-foot Gem.

[Load per stroke, 450 foot-pounds.]

Wind velocity per hour.	Revolutions of wind wheel per minute.	Stroke of pump per minute.	Gallons pumped per minute.	Useful horse-power.
12 miles	12.4	6.2	18.6	0.085
16 miles	23.8	11.9	35.7	0.162
20 miles	29.4	14.7	44.1	0.201
25 miles	32.0	16.0	48.0	0.219

This curve (fig. 18) shows that a 9-mile wind is necessary to start this mill, and that the greatest number of strokes per mile of wind

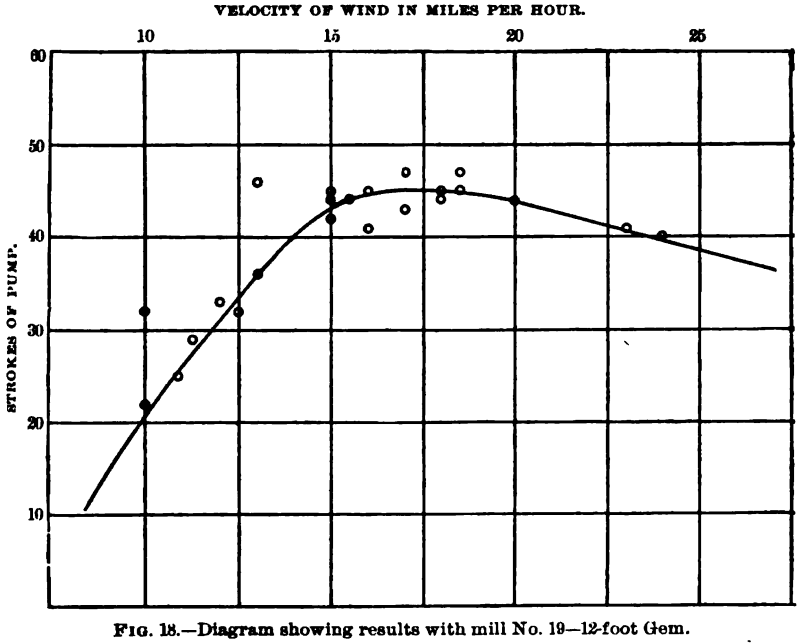


FIG. 18.—Diagram showing results with mill No. 19—12-foot Gem.

is 45—about 25 per cent less than most 12-foot steel back-geared mills, being at the rate of 6 strokes at 12 miles and 16 strokes at 25 miles an hour. The load of this mill is somewhat greater than that of No. 3, and its power should be equal or greater, but it is seen to be much less.

Mill No. 20.—This is a 15½-foot Jumbo (shown in Pl. X, *A*). Its axis is a steel shaft 8 feet above the ground. It has 6 sails, each 9½ by 3½ feet, with an outer radius of 7¾ feet and an inner radius of 4½ feet. This mill operates two pumps, one at each end of the axis, each pump having a 6-inch cylinder, a 3-inch discharge pipe, and a 3-inch supply pipe on a well point 5 feet long. The discharge per stroke of the two pumps was 10 quarts, the lift about 14 feet. The mean barometric pressure was 27.09 inches, the mean temperature 85° F. The anemometer was held 14 feet above the surface of the ground, or at the elevation of the center of a sail when in its highest position. The wheel is set in a large box the top of which is on a level with the axis of the wheel, to prevent the wind from striking the part of the wheel below its axis. The results of the tests are as follows:

Results of tests of mill No. 20—15½-foot Jumbo.

[Load per stroke, 291.2 foot-pounds.]

Wind velocity per hour.	Revolutions of wind wheel per minute.	Strokes of pump per minute.	Gallons pumped per minute.	Useful horse-power.
16 miles	5.3	5.3	13.2	0.047
20 miles	10.7	10.7	26.7	0.095

The curve shown in fig. 19 is seen to be different from the others in that it apparently has no maximum. It starts at a wind velocity of 13

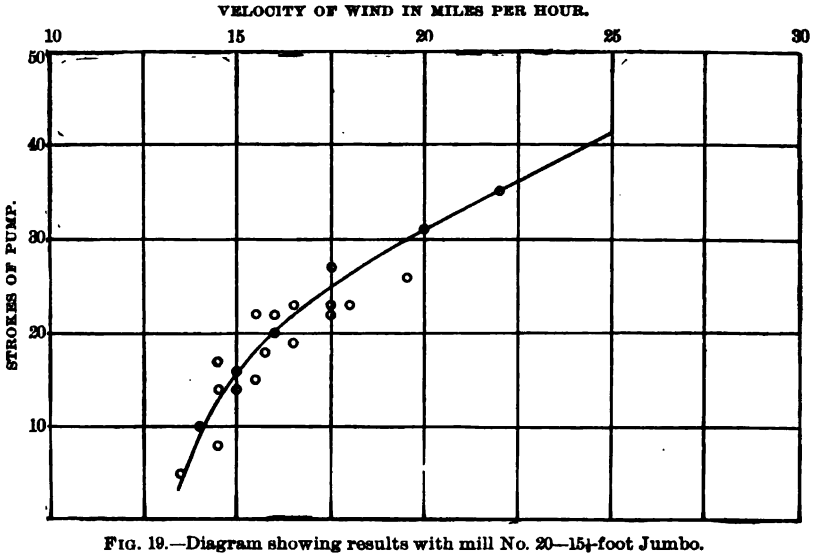


FIG. 19.—Diagram showing results with mill No. 20—15½-foot Jumbo.

to 14 miles, and beyond 20 miles it appears to be a nearly straight line. Comparing these results with those of No. 5, it will be seen that this mill is inferior in power to a good 8-foot steel mill.

A. VIEW OF MILL NO. 20—15½-FOOT JUMBO.

B. VIEW OF DEFENDER MILL AND PUMP KNOWN AS "WATER ELEVATOR."

VIEW OF MILL NO. 21—12-FOOT HALLIDAY.

Mill No. 21.—This is a 12-foot Halliday (shown in Pl. XI) on a 31-foot wooden tower. It was made by the United States Wind Engine and Pump Company, of Batavia, Illinois. The working parts are

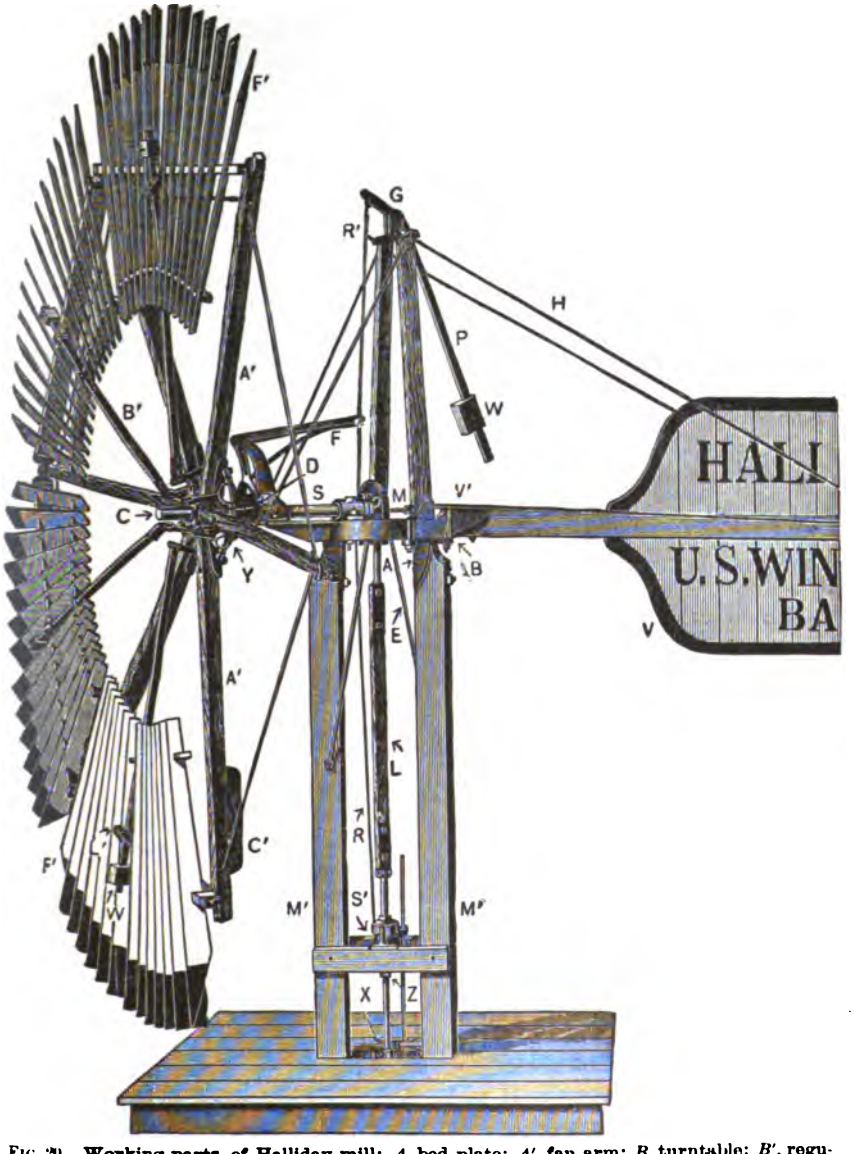


FIG. 20.—Working parts of Halliday mill: *A*, bed plate; *A'*, fan arm; *B*, turntable; *B'*, regulating rod; *C*, front plate; *C'*, assisting weight; *D*, sliding head; *E*, tie rod; *F*, forked lever; *F'*, fans or sails; *G*, truss frame; *H*, truss rod; *L*, pitman; *L'*, fan lever; *M*, crank plate; *M' M'*, masts; *P*, weight lever; *R*, shut-off rod; *R'*, shut-off rod lever; *S*, main shaft; *S'*, sleeve; *V*, vane; *V'*, vane arm; *W*, weight; *W'*, counterweight; *X*, swivel box; *Y*, spider; *Z*, sliding box.

shown in fig. 20. The wheel has 64 sails, each $42\frac{1}{2}$ by 5 by $2\frac{3}{8}$ inches, set at an angle of 35° with the plane of the wheel. It is not back-geared, and regulates itself on the centrifugal principle—the sails

taking the direction of the wind. The pump is of the Stone make, with 7½-inch discharge pipe, 4-inch supply pipe, and 7 inches stroke. The check valve is of the lift variety, the plunger valve of the double-flap variety. The well, which is open, is formed by a wooden curb, 12 inches in diameter, sunk in the bottom of a dug well 9 feet deep. The depth to water was 11½ feet and the lift 11 feet. The discharge per stroke was 4½ quarts when pumping quite rapidly (30 strokes per minute). The valves were not in very good repair and the pump lost its priming after a time. The results of the tests are as follows:

Results of tests of mill No. 21—12-foot wooden Halliday.

[Load per stroke, 141.5 foot-pounds.]

Wind velocity per hour.	Revolutions of wind wheel per minute.	Strokes of pump per minute.	Gallons pumped per minute.	Useful horse-power.
12 miles	14.0	14.0	15.9	0.060
16 miles	28.5	28.5	32.1	0.121
20 miles	37.3	37.3	42.0	0.159
25 miles	44.6	44.6	50.2	0.184

This curve (shown in fig. 21), which is for a lightly loaded (141.5 foot-pounds) direct-stroke 12-foot mill, will be seen to start at a wind

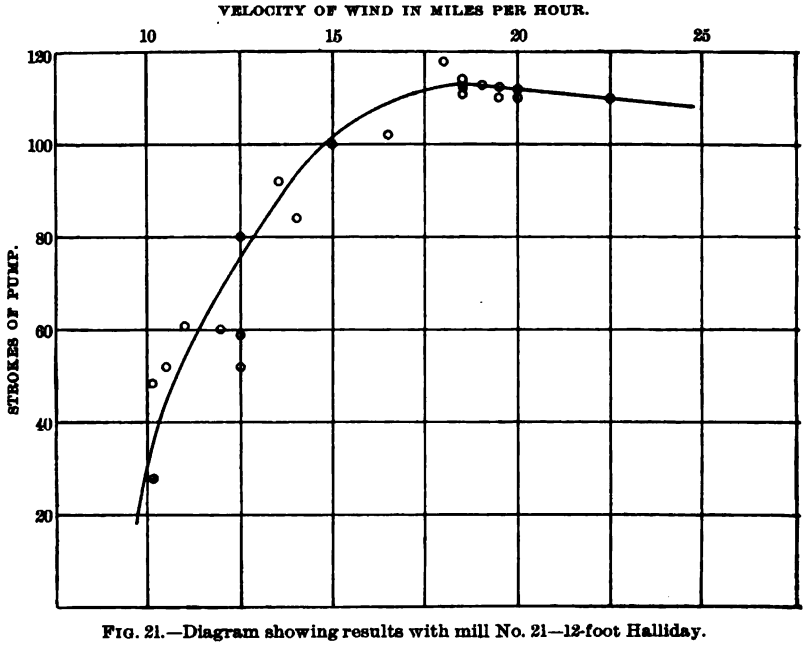


FIG. 21.—Diagram showing results with mill No. 21—12-foot Halliday.

velocity of 9 to 10 miles and to reach a maximum at 19 miles, with a speed of 112 strokes per mile. At 30 miles the number of strokes is

about 98 per mile. The number of strokes per minute varies from 14 at 12 miles to 45 at 25 miles. The power of this mill is only about half that of the 12-foot Aermotor, No. 3.

Mill No. 25.—This is an 8-foot steel mill on a 32-foot steel tower, made by Fairbanks, Morse & Company. The wheel has 18 curved sails, each 29 by 11⅝ by 5¼ inches, set at an angle of 29° with the plane of the wheel. It is back-geared, 2½ to 1. The pump is of the common hand variety, with a 2½-inch cylinder, 1½-inch supply and discharge pipes, and 4 inches stroke. The well is open, 6¼ feet to water. The discharge per stroke was 0.31 quart and the lift 8½ feet. The water raised is used for watering stock. The results of the tests are as follows:

Results of tests of mill No. 25—8-foot Fairbanks-Morse steel mill.

[Load per stroke, 5.5 foot-pounds.]

Wind velocity per hour.	Revolutions of wind wheel per minute.	Strokes of pump per minute.	Gallons pumped per minute.	Useful horse-power.
8 miles	30.7	12.3	1.0	0.002
12 miles	73.0	29.2	2.3	0.005
16 miles	93.2	37.3	2.9	0.007
20 miles	95.0	38.0	3.0	0.008
25 miles	83.2	33.3	2.6	0.005

This is a very lightly loaded mill—only 5.5 foot-pounds per stroke of pump. The number of strokes per minute in light winds is large; the number of strokes per minute in a 25-mile wind is only 33.3, compared with 38 in a 20-mile wind. A comparison of this mill with No. 5 will show the difference between a small pumping outfit for stock purposes and one for irrigation.

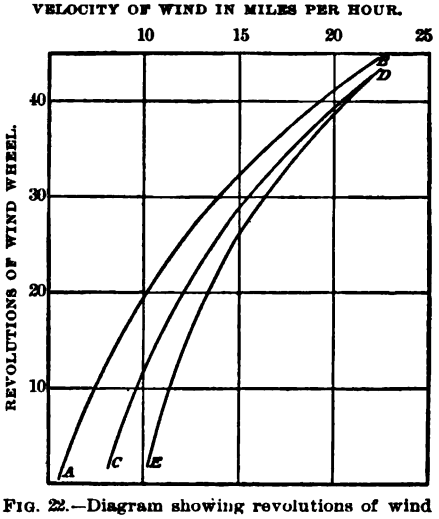


FIG. 22.—Diagram showing revolutions of wind wheel of mill No. 30—16-foot Irrigator. *AB* is for a load of 32 foot-pounds per revolution of wind wheel; *CD* is for a load of 251 foot-pounds; *ED* is for a useful pump load of 337 foot-pounds.

Mill No. 30.—This is a 16-foot Irrigator manufactured by M. Schow, of Kinsley, Kansas. It is a power mill, but not geared forward, and works a pump called a "water elevator." The tower is of wood, 22 feet to axis of wheel. The wind wheel has 10 plane wooden sails, each 70½ by 16 by 13½ inches, set at an angle of 39° to the direction of the wind. The vertical shafting is geared back 30 to 13, and the horizontal shaft is geared forward 13 to 30. The water is lifted from a well 8 feet

by 10 feet (7 feet depth to water), cased with wood. The buckets of the elevator are made of galvanized iron and are 14 by 7½ by 5½ inches, set 18 inches center to center, and hold 3 gallons each. Each bucket has a valve in its bottom. The lift is about 11 feet, and there are 20 buckets on the elevator chains. There is a box in the bottom of the well into which the buckets dip to get their supply of water. This box has a screen in its bottom to keep out the sand. The mill does not govern well, on account of side draft. The mean barometric pressure was 27.7 inches, the mean temperature 72° F. The results of the tests are as follows:

Results of tests of mill No. 30—16-foot Irrigator.

Load on brake.	Load per revolution of wind wheel.	Number of revolutions of wind wheel per minute at given wind velocities (per hour).					Useful horsepower at given wind velocities (per hour).				
		8 miles.	12 miles.	15 miles.	20 miles.	25 miles.	8 miles.	12 miles.	15 miles.	20 miles.	25 miles.
Pounds.	*Ft.-lbs.*										
2	32	12	26	33	41	44	0.012	0.025	0.032	0.040	0.043
16	251	.....	19	29	39	42	.....	0.140	0.220	0.300	0.320
Pump.	337	.....	14	26	38	42	.....	0.140	0.250	0.400	0.440

Fig. 22 shows the number of revolutions per minute of the wind wheel for three loads. Fig. 23 shows the horsepower for these loads.

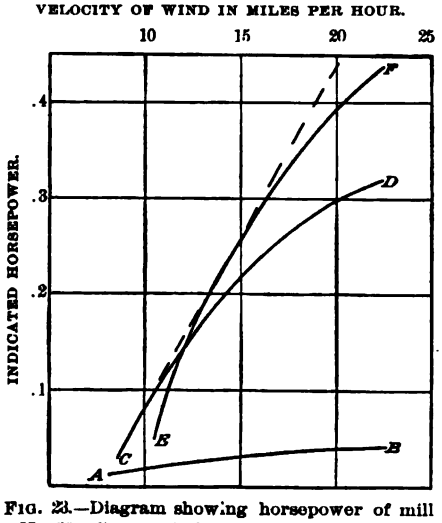


FIG. 23.—Diagram showing horsepower of mill No. 30. Curve *AB* shows the horsepower for a 32 foot-pound load; *CD*, the power for a 251 foot-pound load; *EF*, the power for a 337 foot-pound load; dotted curve shows maximum horsepower for best load.

Comparing the useful pump horsepower of this mill with that of the 16-foot mill No. 9, it will be seen that this mill is not so powerful as No. 9. The power of the Aermotor for low velocities is much greater than that of the Irrigator.

Mill No. 31.—This is a 14-foot Elgin wooden power mill, manufactured at Elgin, Illinois. It works a rotary (Wonder) pump. The wind wheel is on a 48-foot wooden tower. It has 88 plane wooden sails, each 52 by 6 by 3⅛ inches, set at an angle of 37° to the plane of the wheel. It is a sectional vaneless wheel; in place of the vane there is a heavy counterpoise. The wheel is geared forward about 7.19 to 1. The rotary pump is a 3 inch. It is on a well point 6 inches in diameter and 8 feet long; the point penetrates the water to a depth of 8 feet.. The lift was about 18 feet. The suction and discharge pipes

are each 3 inches in diameter. The discharge was 2 quarts per revolution of pump. The pump is manufactured by the National Pump Company, of Kansas City, Missouri. In a 15-mile to a 20-mile wind the windmill worked the pump before priming, but would not start it in that wind after priming. After several attempts of the mill to start the pump the pulley turned on the shaft so that it could not be used. It was very evident that the pump was too great a load for the mill. The owner stated that the mill would only run during a strong wind.

Mill No. 32.—This mill is like the 12-foot Aermotor, the working parts of which are shown in fig. 7. It is on a 40-foot steel tower. The wheel has 18 curved sails, each 44 by 18¾ by 7¾ inches, set at an angle of 31° to the plane of the wheel. The pump is of the Woodmanse type, a sectional view of which is shown in fig. 5, and has an 8-inch cylinder and 12 inches stroke. It is in an open well. The depth to water was 14 feet, the lift 20 feet, and the discharge per stroke 7 quarts. It is back-geared, 3⅓ to 1. The water is pumped into a pond and used for irrigation. The mean barometric pressure was 27.9 inches, the mean temperature 82° F. The results of the tests are as follows:

Results of tests of mill No. 32—12-foot Aermotor.

[Load per stroke, 313 foot-pounds.]

Wind velocity per hour.	Revolutions of wind wheel per minute.	Strokes of pump per minute.	Gallons pumped per minute.	Useful horse-power.
8 miles	17.7	5.3	9.3	0.047
12 miles	49.3	14.8	25.9	0.131
15 miles	60.0	18.5	32.4	0.164
20 miles	73.0	22.0	38.5	0.195

Mill No. 33—This is a 10-foot Woodmanse pumping mill on a 40-foot steel tower. The working parts are like those shown in fig. 4. The wheel has 24 curved sails, each 30½ by 12½ by 5½ inches, set at an angle of 29° to the plane of the wheel. It is back-geared, 2½ to 1. The pump is of the Woodmanse make (like that shown in fig. 5), with a 6-inch cylinder and 10 inches stroke. The depth to water was 14 feet. The pump is on a well point 3½ inches in diameter, 4 feet long, and 67 feet below the surface of the ground. The suction pipe is 3½ inches in diameter, the discharge pipe 6 inches in diameter. The lift was about 22 feet, the discharge per stroke 3¾ quarts. The water is pumped into a pond and used for irrigation. The mean barometric pressure was 27.9 inches, the mean temperature 88° F.

The results of the tests are as follows:

Results of tests of mill No. 33—10-foot Woodmanse.

[Load per stroke, 173 foot-pounds.]

Wind velocity per hour.	Revolutions of wind wheel per minute.	Strokes of pump per minute.	Gallons pumped per minute.	Useful horse-power.
8 miles	8.7	3.5	3.3	0.020
12 miles	36.0	14.4	13.5	0.075
15 miles	51.2	20.5	19.2	0.110

Mill No. 35.—This is an 8-foot steel Dempster pumping mill, manufactured by the Dempster Manufacturing Company, of Beatrice, Nebraska, and is shown in Pl. XII. The tower is of steel, 30 feet to axis of wheel. The wind wheel has 18 curved sails, each 30½ by 14½ by 7 inches, set at an angle of 27° to the plane of the wheel. It is back-geared, 3 to 1. The well is an open dug well. The distance from the surface of the ground to the water was 39 feet. The water is pumped into a tank 22 feet above ground, and is used for irrigation. The pump has an 8-inch stroke, a 3¼-inch cylinder, and 2-inch suction and discharge pipes. The lift was 58 feet, the discharge per stroke 1.1 quarts. The mean temperature was 51° F., the mean barometric pressure 28.9 inches. The results of the tests are as follows:

Results of tests of mill No. 35—8-foot steel Dempster.

[Load per stroke, 133.5 foot-pounds.]

Wind velocity per hour.	Revolutions of wind wheel per minute.	Strokes of pump per minute.	Gallons pumped per minute.	Useful horse-power.
12 miles	57.6	19.2	5.3	0.077
16 miles	86.4	28.8	7.9	0.116
20 miles	99.9	33.3	9.2	0.134

In a 15-mile wind the pump made 26 to 27 strokes per minute, and when the pump rod was uncoupled from the pump it made 34 strokes per minute in the same wind. Fig. 24 shows the number of strokes per minute for different wind velocities. This mill is heavily loaded—133 foot-pounds per stroke of pump. It starts at a wind velocity of about 9 miles an hour, and as the wind increases the number of strokes increases rapidly at first, then more slowly. At 25 miles an hour the number of strokes is 37 per minute. This mill is back-geared, 3 to 1, so that in a 25-mile wind the wind wheel makes 111 revolutions per minute and has a circumference velocity of 46.5 feet per second.

Mill No. 36.—This is a 22½-foot Eclipse wooden pumping mill, lift-

VIEW OF MILL NO. 35—8-FOOT DEMPSTER.

ing water into a tank for railroad purposes. The tower is of wood, 52 feet to the axis of the wheel. The wind wheel has 136 plane sails, each 105 by 6 by 2 inches, set at an angle of 39° to the plane of the wheel. It works by direct stroke. The well, which is open, is 20 feet in diameter; the depth to water was 19 feet from the surface of the ground. The pump is double-acting, and has a 4½-inch cylinder, 7 inches stroke, and 2-inch suction and discharge pipes. The lift was 39 feet. The tank is 90 feet from the well. The discharge could not be measured, but as the packing of the pump was new it was approximately equal to twice the volume of the cylinder, or 0.76 gallon per double stroke. The mean temperature was 58° F., the mean barometric pressure 29.43 inches. The results of the tests are as follows:

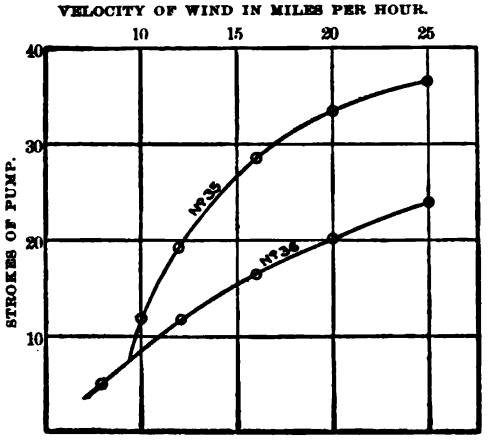


FIG. 24.—Comparative diagram showing results with mills No. 35 (8-foot Dempster) and No. 36 (22½-foot Eclipse).

Results of tests of mill No. 36—22½-foot wooden Eclipse.

[Load per stroke, 248 foot-pounds.]

Wind velocity per hour.	Revolutions of wind wheel per minute.	Strokes of pump per minute.	Gallons pumped per minute.	Useful horse-power.
8 miles	4.8	4.8	3.8	0.036
12 miles	12.0	12.0	9.4	0.090
16 miles	16.5	16.5	12.8	0.124
20 miles	20.0	20.0	15.6	0.150
25 miles	24.2	24.2	18.8	0.182

Fig. 24 shows the strokes per minute at different wind velocities. The diagram is seen to be quite different from that of mill No. 35. The mill starts at a wind velocity of 6 or 7 miles an hour, and increases gradually to 24 strokes in a 25-mile wind. The circumference velocity of the latter is 46.5 feet per second, that of the former 29 feet. The wind wheel of this mill is making 24 revolutions per minute; that of No. 35 is making 111 revolutions per minute in a 25-mile wind.

Mill No. 37.—This is a 12-foot steel Woodmanse Mogul like that shown in fig. 4. It pumps water into a pressure tank 9½ feet by 2½ feet in a cellar and about 170 feet from the well. The pump and pressure

tank are shown in fig. 25. The tower is of steel, and is 40 feet to the axis of the wheel. The wind wheel has 30 curved sails, each 37 by 13 by 5.5 inches, set at an angle of 29° to the plane of the wheel. The well is 50 feet deep and 8 inches in diameter; the depth to water was 42 feet. The pump has a 3-inch cylinder and 9 inches stroke. The discharge pipe is 1 inch in diameter, the suction pipe 1¼ inches in

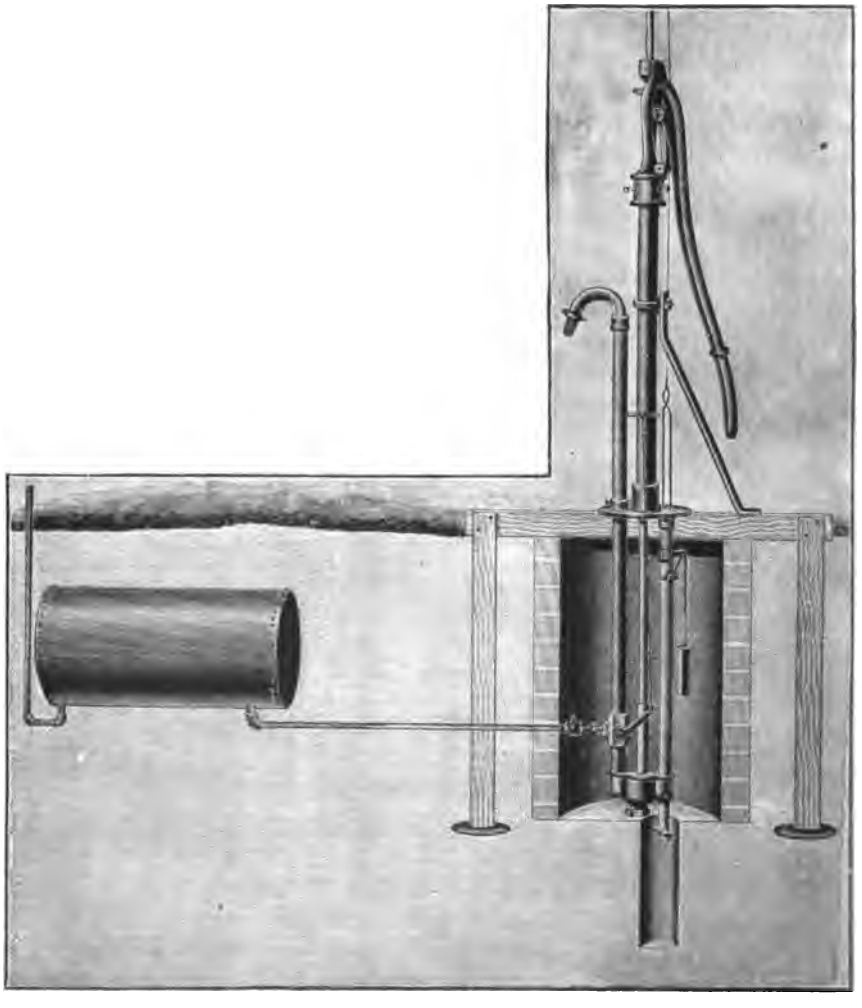


FIG. 25.—Pump, pressure tank, and hydraulic regulator of mill No. 37—12-foot Woodmanse Mogul.

diameter. The discharge per stroke was 1.05 quarts. The load on the pump was equal to 43 feet head of water (the friction in about 200 feet of 1-inch pipe) and a compressed-air pressure the amount of which was recorded on a gage. The mean temperature was 66° F., the mean barometric pressure 29.1 inches. The results of the tests are as follows:

Results of tests of mill No. 37—12-foot steel Woodmanse Mogul.

Load per stroke.	Number of strokes of pump per minute at given wind velocities (per hour).						Useful horsepower at given wind velocities (per hour).					
	8 miles.	12 miles.	16 miles.	20 miles.	25 miles.	30 miles.	8 miles.	12 miles.	16 miles.	20 miles.	25 miles.	30 miles.
Ft.-lbs.												
94	7.7	18.4	24.5	29.3	34.2	38.0	0.022	0.053	0.071	0.083	0.097	0.108
254	.....	12.2	19.0	23.3	26.7	29.0	.....	0.094	0.146	0.180	0.205	0.223
350	.....	.....	.....	18.0	21.0	21.5	.....	.....	.....	0.191	0.223	0.228
473	.....	.....	.....	10.7	.....	.....	.....	.....	.....	0.153	.....	.....

Fig. 26 shows the number of strokes per minute for different wind velocities for four useful pump loads, viz, 43 feet, 43 feet plus 32 pounds, 43 feet plus 50 pounds, and 43 feet plus 75 pounds, or, reducing the pounds pressure to head in feet, the four useful loads are: 43 feet for the curve *aa'*, 116 feet for the curve *bb'*, 160 feet for the curve *cc'*, and 216 feet for the curve *dd'*. The effect of increased load on the number of strokes is well shown here. The effect of the hydraulic regulator may be seen in the curve *cc'*, but it is shown to a greater extent in the curve *dd'*.

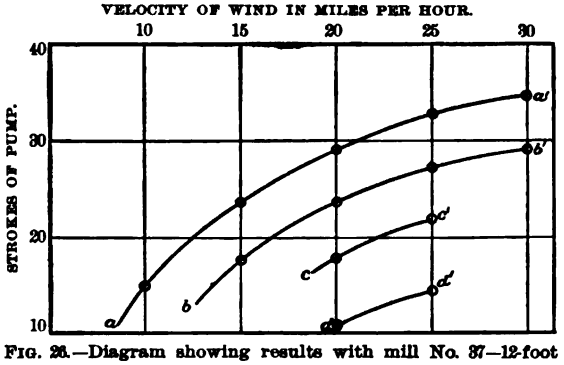


FIG. 26.—Diagram showing results with mill No. 37—12-foot Woodmanse Mogul. The curve *aa'* is for a useful pump load of 43 feet head; *bb'* is for a useful pump load of 116 feet head; *cc'* is for a useful pump load of 160 feet head; *dd'* is for a useful pump load of 216 feet head. The effect of the hydraulic regulator is shown in the curves *cc'* and *dd'*.

Mill No. 38.—This is a 10-foot Woodmanse wooden mill on a 30-foot wooden tower, and is used to pump water for stock. The wind wheel has 96 plane sails, each 34 by 3½ by 1¼ inches, set at an angle of 38° to the plane of the wheel. The well is driven and is 107 feet deep; the depth to water was about 44 feet. The pump works direct and has a 2½-inch cylinder and 4 inches stroke. The lift was about 50 feet, and the discharge per stroke of pump $\frac{1}{10}$ gallon.

The results of the tests are as follows:

Results of tests of mill No. 38—10-foot wooden Woodmanse.

[Load per stroke, 21 foot-pounds.]

Wind velocity per hour.	Revolutions of wind wheel per minute.	Strokes of pump per minute.	Gallons pumped per minute.	Useful horse-power.
8 miles	18	18	0.9	0.012
12 miles	29	29	1.5	0.019
16 miles	36	36	1.8	0.023
20 miles	41	41	2.1	0.026
25 miles	46	46	2.3	0.029

Fig. 27 shows the number of strokes per minute for different wind velocities. This is a lightly loaded mill, working direct stroke. It

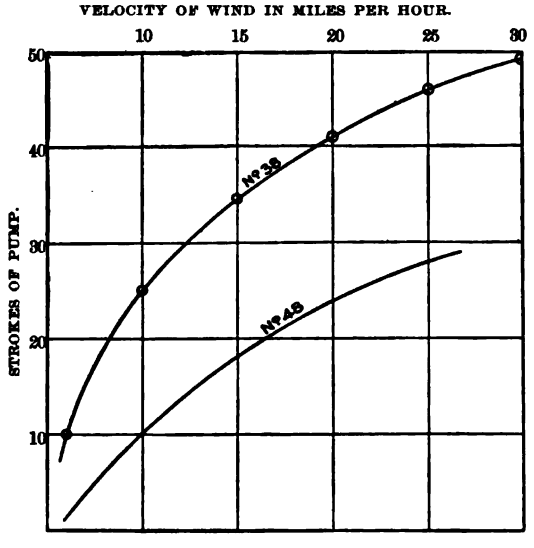


FIG. 27.—Comparative diagram showing results with mills No. 38 (10-foot wooden Woodmanse) and No. 48 (30-foot wooden Halliday).

starts in a 5-mile wind; the number of strokes increases very rapidly at first, and more slowly for high velocities. In a 30-mile wind the number of strokes per minute is 50. The circumference velocity of this wheel in a 25-mile wind is 24 feet per second.

Mill No. 39.—This is a 10-foot Woodmanse direct-stroke iron pumping mill, used to pump water for stock. The tower is of iron, 35 feet to the axis of the wheel. The wind wheel has 18 curved sails, each 30 by 12½ by 7¾ inches, set at an angle of 40° to the plane of the wheel. The pump is on a well point 24 feet below the surface of the ground. It has a stroke of 6 inches, a 3½-inch cylinder, and 1½-inch suction and discharge pipes. The lift was about 27 feet, the discharge per stroke 1 quart. The results of the tests are as follows:

Results of tests of mill No. 39—10-foot iron Woodmanse.

[Load per stroke, 36 foot-pounds.]

Wind velocity per hour.	Revolutions of wind wheel per minute.	Strokes of pump per minute.	Gallons pumped per minute.	Useful horse-power.
8 miles	15	15	3.7	0.017
12 miles	30	30	7.5	0.034
16 miles	40	40	10.0	0.044
20 miles	47	47	11.7	0.052

Mill No. 40.—This is an 8-foot steel pumping mill manufactured by Fairbanks, Morse & Company. It is used to pump water for stock. It is on a 30-foot steel tower. The wind wheel has 18 curved sails, each 29 by 10½ by 5 inches, set at an angle of 29° to the plane of the wheel. It is back-geared, 2½ to 1. The pump is on a well point 20 feet below the surface of the ground. It has a stroke of 6 inches, a 3½-inch cylinder, and 1½-inch suction and discharge pipes. The lift was about 22 feet, and the discharge per stroke $\frac{1}{15}$ gallon. The results of the tests are as follows:

Results of tests of mill No. 40—8-foot Fairbanks-Morse steel mill.

[Load per stroke, 13 foot-pounds.]

Wind velocity per hour.	Revolutions of wind wheel per minute.	Strokes of pump per minute.	Gallons pumped per minute.	Useful horse-power.
8 miles	32.5	13.0	0.8	0.005
12 miles	55.0	22.0	1.5	0.009
16 miles	72.0	29.0	1.9	0.012
20 miles	85.0	34.0	2.3	0.014

Mill No. 41.—This is a 12-foot Woodmanse direct-stroke iron pumping mill, used for pumping water for stock. The tower is of iron, 30 feet to the axis of the wheel. The pump has a 3-inch cylinder and 6 inches stroke. The wind wheel has 24 curved sails, each 36 by 14½ by 7¾ inches, set at an angle of 39° to the plane of the wheel. The lift was 28 feet, and the discharge per stroke ⅛ gallon. The results of the tests are as follows:

Results of tests of mill No. 41—12-foot iron Woodmanse.

[Load per stroke, 29 foot-pounds.]

Wind velocity per hour.	Revolutions of wind wheel per minute.	Strokes of pump per minute.	Gallons pumped per minute.	Useful horse-power.
8 miles	14	14	1.7	0.012
12 miles	25	25	3.1	0.022
16 miles	33	33	4.1	0.029
20 miles	40	40	5.0	0.036

Mill No. 42.—This is a 6-foot Ideal pumping mill on a 22-foot steel tower, and is used for pumping water for stock. It is shown in Pl. XIII. The wheel has 12 curved sails, each 26 by 14½ by 5 inches, set at an angle of 39° to the plane of the wheel. It is back-geared, 4 to 1. The well is a dug well; depth to water, 15 feet. The pump has a 3-inch cylinder and 6 inches stroke. The lift was 16 feet, the discharge per stroke 1 quart. The results of the tests are as follows:

Results of tests of mill No. 42—6-foot Ideal.

[Load per stroke, 35 foot-pounds.]

Wind velocity per hour.	Revolutions of wind wheel per minute.	Strokes of pump per minute.	Gallons pumped per minute.	Useful horse-power.
8 miles	16	4	1.0	0.004
12 miles	52	13	3.2	0.014
16 miles	76	19	4.7	0.020
20 miles	92	23	5.7	0.024

FIG. 28.—View of mill No. 43—10-foot Perkins.

This is the smallest mill yet tested. It is rather heavily loaded for its size; in fact, it is more heavily loaded than the 12-foot mill No. 41. It is doing very good work for a mill of its size.

VIEW OF MILL NO. 42—6-FOOT IDEAL.

Mill No. 43.—This is a 10-foot Perkins direct-stroke pumping mill on a 32-foot steel tower, and is used to pump water for stock. The working parts are shown in fig. 28. The wind wheel has 30 curved sails, each 30 by 10½ by 5 inches, set at an angle of 29° to the plane of the wheel. The pump is on a well point 34 feet below the surface of the ground. It has a 3-inch cylinder and 4 inches stroke. The lift was about 36 feet, the discharge per stroke ½ quart. The exposure was not good. The results of the tests are as follows:

Results of tests of mill No. 43—10-foot Perkins.

[Load per stroke, 37 foot-pounds.]

Wind velocity per hour.	Revolutions of wind wheel per minute.	Strokes of pump per minute.	Gallons pumped per minute.	Useful horse-power.
8 miles	20	20	2.5	0.022
12 miles	32	32	4.0	0.035
16 miles	41	41	5.1	0.045
20 miles	45	45	5.6	0.050

Mill No. 45.—This is a 10-foot Eclipse direct-stroke pumping mill used to pump water for stock. The tower is of wood, 40 feet to the axis of the wheel. The wind wheel has 84 plane sails, each 36½ by 4 by 1½ inches, set at an angle of 35° to the plane of the wheel. The well is a dug well; depth to water, 12 feet. The pump has a stroke of 6 inches, a 3-inch cylinder, and 1½-inch suction and discharge pipes. The lift was 14 feet, and the discharge per stroke 0.62 quart. The results of the tests are as follows:

Results of tests of mill No. 45—10-foot wooden Eclipse.

[Load per stroke, 18 foot-pounds.]

Wind velocity per hour.	Revolutions of wind wheel per minute.	Strokes of pump per minute.	Gallons pumped per minute.	Useful horse-power.
8 miles	18	18	2.8	0.010
12 miles	28	28	4.3	0.014
16 miles	32	32	5.0	0.016
20 miles	35	35	5.4	0.018
25 miles	38	38	5.9	0.019

Mill No. 46.—This is a 10-foot Cornell direct-stroke wooden mill manufactured at Louisville, Kentucky, and is used to pump water for stock. The tower is of wood, 30 feet to the axis of the wheel. The wind wheel has 90 plane sails, each 36 by 4½ by 1½ inches, set at an angle of 47° to the plane of the wheel. The well is a dug well; the water is only about 3 feet below the surface of the ground. The pump has a 3½-inch cylinder and 4 inches stroke. The discharge

per stroke was 1 pint when pumping rapidly. The lift was 5⅔ feet. The results of the tests are as follows:

Results of tests of mill No. 46—10-foot wooden Cornell.

[Load per stroke, 6 foot-pounds.]

Wind velocity per hour.	Revolutions of wind wheel per minute.	Strokes of pump per minute.	Gallons pumped per minute.	Useful horse-power.
8 miles	16	16	2.0	0.003
12 miles	25	25	3.1	0.004
16 miles	36	36	4.5	0.006
20 miles	42	42	5.2	0.007
25 miles	48	48	6.0	0.007
30 miles	52	52	6.5	0.008

Mill No. 47.—This is a 10-foot Dempster steel mill, like that shown in Pl. XII, on a 40-foot steel tower. The wind wheel has 24 curved sails, each 30½ by 13½ by 6½ inches, set at an angle of 29° to the plane of the wheel. The pump is back-geared, 2⅞ to 1, and has a 3-inch cylinder and 7 inches stroke. The well is open, 20¾ feet to water, and situated under the porch of a house. The mill is located 76 feet from the well. The cylinder is directly under the mill, in a chamber 4 feet by 4 feet, and 6 feet deep. The lower end of the cylinder was about 12 feet vertically above the surface of the water in the well. The discharge per stroke was 1 quart when pumping rapidly. The lift was about 18 feet. The wind wheel of this mill is not well balanced, and the spring which holds it in the wind is not stiff enough. The results of the tests are as follows:

Results of tests of mill No. 47—10-foot steel Dempster.

[Load per stroke, 37 foot-pounds.]

Wind velocity per hour.	Revolutions of wind wheel per minute.	Strokes of pump per minute.	Gallons pumped per minute.	Useful horse-power.
8 miles	37	13	3.2	0.014
12 miles	66	23	5.7	0.026
16 miles	89	31	7.8	0.035
20 miles	95	33	8.3	0.037
25 miles	0	0	0	0.000

A diagram which was platted for this mill shows that the curve drops at about 19 miles an hour, and reaches the axis line at 24 miles. Above 24 miles an hour the wheel is entirely out of the wind and does no work. On this diagram was also platted the number of strokes per minute of the 8-foot Dempster (No. 35), to show the effect of the different loads upon the number of strokes per minute. The 8-foot mill was found to be carrying more than three times the load that the 10-foot mill was carrying.

VIEW OF MILL NO. 48—30-FOOT HALLIDAY.

Mill No. 48.—This is a 30-foot Halliday wooden pumping mill on a 70-foot wooden tower. It is owned by the city of Valley Falls, Kansas, and is used to pump water for the city supply. The mill and tower are shown in Pl. XIV. The sail area is arranged in two concentric rings. In the outer ring there are 192 sails, in the inner ring 144 sails, each 43 by 4½ by 3½ inches, set at an angle of 25° to the plane of the wheel. There are two wells; one (11 feet in diameter) directly under the mill, and another (10 feet in diameter) near the bank of the river 375 feet from the mill. A 3-inch suction pipe connects the wells, and a 3-inch supply pipe leads from the lower well to the river. The pump is double-acting, and has a 4-inch cylinder and 11 inches stroke. The water is pumped directly into the distribution pipes, also into an elevated tank. The tank is of wood, 20 feet by 30 feet, and is 5,570 feet distant from the mill. Of the connecting pipe, 50 feet is 3 inches in diameter, 1,200 feet is 4 inches in diameter, and 4,300 feet is 6 inches in diameter. The bottom of the tank is 111 feet above the well platform. The lift or head at any time is, then, the distance from the well to the well platform and 111 feet plus the amount registered on the gage on the tank. The mean lift when the test was made was 135 feet. The cylinder capacity is 2.4 quarts, the measured discharge per double stroke 4.5 quarts. The mean temperature was 90° F., the mean barometric pressure 28.9 inches. The results of the tests are as follows:

Results of tests of mill No. 48—30-foot wooden Halliday.

[Load per stroke, 1,265 foot-pounds.]

Wind velocity per hour.	Revolutions of wind wheel per minute.	Strokes of pump per minute.	Gallons pumped per minute.	Useful horse-power.
8 miles	6	6	6.7	0.23
12 miles	15	15	16.9	0.58
16 miles	20	20	22.5	0.77
20 miles	24	24	27.0	0.92
25 miles	28	28	31.5	1.07

This windmill and pump had been in use about ten years. It furnishes enough water during nine months in the year, but during the months of July and August and part of June and September a steam engine is at times employed to work the pump. The cost of repairs to the mill and pump has been from $50 to $60 a year. The number of strokes per minute for different wind velocities is shown in fig. 27.

This is the largest windmill pumping outfit that we have tested. It is interesting to compare its power with that of the smaller wooden mills and with that of the steel mills.

Mill No. 51.—This is an 8-foot Monitor steel pumping mill on a 30-foot steel tower (see fig. 29). The wind wheel has 18 curved sails, each 31½ by 13 by 5 inches, set at an angle of 35° with the plane of

the wheel. The well is a dug well; depth to water, 18½ feet. The water is used for stock. The pump has a 3-inch cylinder, 6 inches stroke, and 1½-inch suction and discharge pipes. The cylinder is 1 foot above the lower end of the suction pipe, and is always under water. A peculiarity of this mill is that the downstroke of the pump

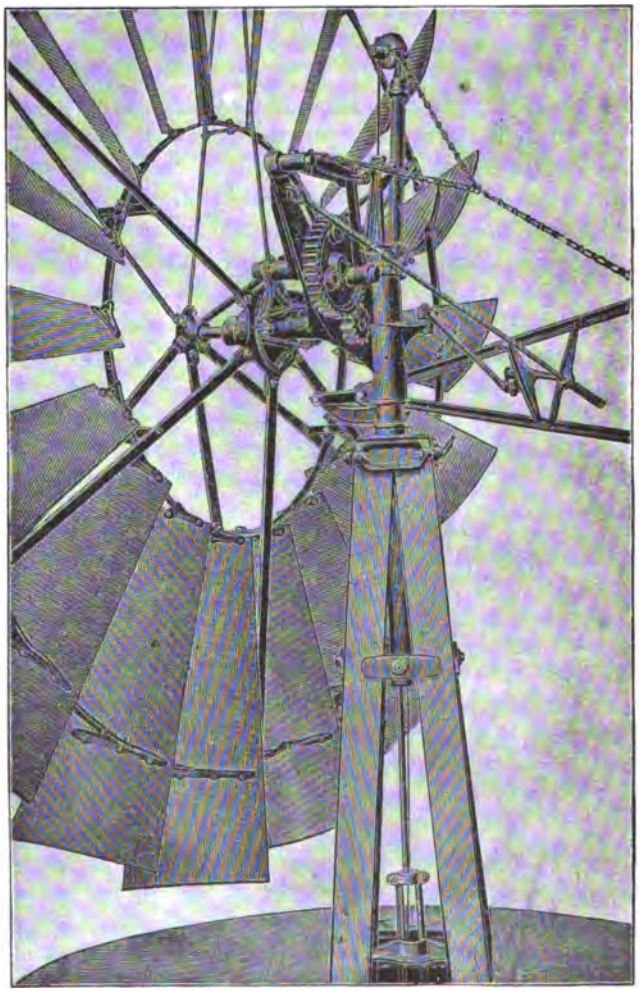


FIG. 29.—Working parts of mill No. 51—8-foot Monitor.

is made in less time than the upstroke. The mill is back-geared, 35 to 13, 13 of the cogs being passed over on the downstroke and 22 on the upstroke. This arrangement makes the mill run easily and prolongs its usefulness. The wind wheel is held in the wind by the weight of the tail; there is no spring. The lift was 25 feet, the discharge per stroke 0.7 quart. The mean temperature was 84° F., the mean

barometric pressure 27.8 inches. The results of the tests are as follows:

Results of tests of mill No. 51—8-foot steel Monitor.

[Load per stroke, 36.5 foot-pounds.]

Wind velocity per hour.	Revolutions of wind wheel per minute.	Strokes of pump per minute.	Gallons pumped per minute.	Useful horse-power.
8 miles	27	10	1.7	0.011
12 miles	56	21	3.7	0.023
16 miles	81	30	5.2	0.034
20 miles	91	34	6.0	0.037

When the mill was running uncoupled from the pump, the pump rod made 24 strokes per minute in a 12-mile wind, i. e., the pump load of 36.5 foot-pounds per stroke reduced the number of the strokes of the pump rod from 24 to 21.

For the purpose of ready comparison the principal results of these tests of pumping mills have been tabulated. (See table on p. 64.)

DISCUSSION OF RESULTS OF TESTS.

In this discussion of the results of the tests of these pumping mills we wish to call attention to the principal facts shown by them. The explanation of some of the points is not easy. The useful work which a windmill will do at a given wind velocity depends on several factors, and it is difficult to measure or even estimate the value of each. If the mills could be tested under conditions easily controlled by the experimenter, the problem would be greatly simplified; but each mill is tested under its own conditions of pump, well, wind exposure, and atmosphere. A comparison of the results of the tests of pumping mills with the results of the tests of power mills throws much light on some of the facts (see Part II, pages 107 to 109, inclusive). A fact very evident from the following table is that the useful work done by windmills in pumping water is small. Only one mill, the largest (No. 48), is doing 1 horsepower of useful work in a 25-mile wind. The best 12-foot mill is doing less than 0.64 horsepower and the best 8-foot mill less than 0.12 horsepower in a 25-mile wind.

Results of tests of pumping mills.

No. of mill.	Name of mill.	Size of mill.	Size of pump. *b*	Lift of pump.	Useful work per stroke.	Number of strokes of pump per minute at given wind velocities (per hour).						Useful horsepower at given wind velocities (per hour).					
						8 miles.	12 miles.	16 miles.	20 miles.	25 miles.	30 miles.	8 miles.	12 miles.	16 miles.	20 miles.	25 miles.	30 miles.
		Feet.	*Inches.*	*Feet.*	*Ft.-lbs.*												
2	Woodmanse	12	9¼x12	17.75	536.2	.....	5.2	16.0	20.3	23.3	25.3	.....	0.085	0.260	0.322	0.379	0.411
3	Aermotor	12	9¼x12	13.75	415.3	5.3	12.0	16.4	19.8	23.1	25.0	0.067	0.151	0.207	0.250	0.291	0.315
4	Ideal	8	5¼x 8	12.00	50.0	.....	10.2	19.3	25.3	28.1	25.0	.....	0.015	0.029	0.038	0.043	0.038
5	Aermotor	8	6 x 8	13.00	94.9	.....	18.6	25.1	29.8	34.6	38.5	.....	0.053	0.072	0.086	0.099	0.111
6	Gem	8	6¼x 8	9.60	77.6	.....	12.4	17.9	22.0	25.4	28.5	.....	0.029	0.042	0.051	0.059	0.065
7	Aermotor	12	9¼x12	15.50	461.9	.....	11.4	15.8	19.0	22.1	23.5	.....	0.160	0.221	0.266	0.309	0.329
8	Star (*a*)	10	3 x 5	30.00	15.0	28.0	30.0	.....	.....	.....	.....	0.013	0.014	.....	.....	.....	.....
9	Aermotor	16	8 x16	44.25	1,013.0	.....	10.6	14.1	17.2	19.6	21.0	.....	0.325	0.433	0.448	0.601	0.644
10	Ideal	8	2¼x 6	33.00	22.8	14.1	25.0	33.3	.....	.....	.....	0.010	0.017	0.023	0.032	.....	.....
11	.....do	12	7¼x12	45.00	843.7	.....	4.8	12.7	18.8	23.3	25.0	.....	0.123	0.325	0.481	0.600	0.639
12	.....do	14	9¼x12	11.00	263.5	3.1	10.9	15.7	19.2	21.5	.....	0.025	0.067	0.125	0.153	0.172	.....
13	Aermotor	12	10 x12	11.00	330.0	.....	11.0	17.1	21.0	24.6	27.5	.....	0.110	0.171	0.210	0.247	0.275
15	Gem	10	8 x 8	15.00	219.0	.....	7.8	11.9	14.7	14.6	.....	.....	0.053	0.082	0.101	0.099	.....
16	Halliday (*a*)	10	6 x 8	16.00	100.0	4.0	22.6	33.9	42.7	52.5	.....	0.012	0.067	0.103	0.130	0.159	.....
17	Gem	12	8 x12	21.75	38.5	.....	6.0	12.8	17.3	.....	.....	.....	0.070	0.149	0.202	.....	.....
18	Ideal	8	5¼x 8	14.75	89.2	8.0	20.2	26.1	28.0	27.5	.....	0.022	0.054	0.070	0.076	0.074	.....
19	Gem	12	10 x10	18.00	450.0	.....	6.2	11.9	14.7	16.0	.....	.....	0.085	0.162	0.201	0.219	.....
20	Jumbo (*a*)	15¼	6 x12	14.00	291.2	.....	.....	5.3	10.7	17.5	.....	.....	.....	0.047	0.095	0.154	.....
21	Halliday (*a*)	12	7¼x 7	15.00	141.5	.....	14.0	28.5	37.3	44.6	.....	.....	0.060	0.121	0.159	0.184	.....
25	Fairbanks	8	2¼x 4	8.50	5.5	12.3	29.2	37.3	38.0	33.3	.....	0.002	0.005	0.007	0.008	0.005	.....
30	Irrigator (*a*)	16	.....	14.00	.....	.....	.....	.....	.....	.....	.....	.....	0.14	0.25	0.40	0.44	.....
32	Aermotor	12	8 x12	20.00	313.0	5.3	14.8	19.0	22.0	.....	.....	0.047	0.131	0.170	0.195	.....	.....
33	Woodmanse	10	6 x10	22.00	172.0	3.5	14.4	20.5	.....	.....	.....	0.020	0.075	0.110	.....	.....	.....
35	Dempster	8	3¼x 8	58.00	183.5	.....	19.2	28.8	33.3	.....	.....	.....	0.077	0.116	0.134	.....	.....
36	Eclipse (*a*)	22¼	4¼x14	39.00	248.0	4.8	12.0	16.5	20.0	24.2	.....	0.036	0.090	0.124	0.150	0.182	.....
37	Woodmanse	12	8 x 9	.....	254.0	.....	12.2	19.0	23.3	26.7	29.0	.....	0.094	0.146	0.180	0.205	0.223
38	.....do. (*a*)	10	2¼x 4	50.00	21.0	18.0	29.0	36.0	41.0	46.0	.....	0.012	0.019	0.023	0.026	0.029	.....
39	.....do	10	3¼x 6	27.00	36.0	15.0	30.0	40.0	47.0	.....	.....	0.017	0.034	0.044	0.052	.....	.....
40	Fairbanks	8	3¼x 6	22.00	13.0	13.0	22.0	29.0	34.0	.....	.....	0.005	0.009	0.012	0.014	.....	.....
41	Woodmanse	12	3 x 6	28.00	29.0	14.0	25.0	33.0	40.0	.....	.....	0.012	0.022	0.029	0.036	.....	.....
42	Ideal	6	3 x 6	16.50	35.0	4.0	13.0	19.0	23.0	.....	.....	0.004	0.014	0.020	0.024	.....	.....
43	Perkins	10	3 x 4	36.00	37.0	20.0	32.0	41.0	45.0	.....	.....	0.022	0.035	0.045	0.050	.....	.....
45	Eclipse (*a*)	10	3 x 6	14.00	18.0	18.0	28.0	32.0	35.0	.....	.....	0.010	0.014	0.016	0.018	.....	.....
46	Cornell (*a*)	10	3¼x 4	5.70	6.0	16.0	25.0	36.0	42.0	48.0	52.0	0.008	0.004	0.006	0.007	0.007	0.008
47	Dempster	10	3 x 7	18.00	37.0	13.0	23.0	31.0	36.0	0	0	0.014	0.026	0.035	0.037	0	0
48	Halliday (*a*)	30	4 x22	135.00	1,265.0	6.0	15.0	20.0	24.0	28.0	.....	0.38	0.58	0.77	0.92	1.07	.....
51	Monitor	8	3 x 6	25.00	36.5	10.0	21.0	30.0	34.0	.....	.....	0.011	0.023	0.034	0.037	.....	.....

a Wooden mills.

b Inside diameter of cylinder by length of stroke.

From the foregoing table it will be seen that mills of the same size differ very much in the amount of useful work, and that some of the larger mills are doing very little more work than some of the smaller ones. Nos. 4 and 18, for example, are the same size and make (the wells, however, are very different), but the latter is doing three or four times more work than the former. The 12-foot mill, No. 11, is doing nearly as much useful work as the 16-foot mill, No. 9, and three or four times more work than the 14-foot mill, No. 12, while it is doing 300 per cent more work than No. 21. The 15½-foot Jumbo will probably do little more work during the season than a good 8-foot mill.

RELATION BETWEEN WIND VELOCITY AND STROKES OF PUMP.

Figs. 6, 10, 11, 12, 13, 15, 17, 18, 19, and 21 show graphically the relation between the wind velocity (in miles per hour) and the strokes of the pump. The curves, as will be noted, differ considerably; but with the exception of fig. 19, for mill No. 20, they agree in that they rise rapidly, reaching the highest point at wind velocities of from 13 to 19 miles. From that point they descend slowly. They differ much in the position of the beginning of the curve, or the velocity required to start the mill. Some will run in an 8-mile wind, while others require a 10-mile or a 12-mile wind to start them. Some rise less rapidly than others, a notable case being mill No. 11. Some descend much more rapidly than others after reaching the highest point. This is especially true of the 8-foot Ideals. Mill No. 20 required a 14-mile wind to start it, and does not appear to have a maximum. The shape of the curve, especially the position of its initial point, is due to the load on the pump, or the number of foot-pounds per stroke. An increase in the load moves the curve to the right and raises it higher. This will be more clearly shown in Part II (see discussion on 12-foot power mill No. 27, pp. 86–89). The height and position of the highest point depend on the tension of the spring, or the weight which holds the mill in the wind. The greater the tension the higher the summit and the farther it is to the right; the less the tension in the spring the steeper the descent from the highest point. The gearing—i. e., the mechanism which causes the pump to make a stroke to each revolution of the wheel, or a stroke every second or third revolution only—modifies the curve. In mills with a direct stroke the curve is much higher and is farther to the right than in back-geared mills, as shown by a comparison of the curves of mills Nos. 3 and 21, shown in figs. 10 and 21.

USEFUL WORK OF PUMPING MILLS.

The relation between wind velocity and horsepower is shown graphically, for five 12-foot mills in fig. 30, and for four 8-foot mills in fig. 31. Examining the five curves of fig. 30, we see that No. 11, the one which gives the greatest horsepower, has the heaviest load and requires the greatest wind velocity to start it. No. 2 has about

five-eighths of the load of No. 11, does less work, and requires about the same wind velocity to start it. No. 3 has a lighter load than No. 2, and will start in a wind of about 7 miles an hour. No. 19 has a little heavier load than No. 3, and does much less work at all velocities. The latter requires a 9-mile or a 10-mile wind to start it, while the former will start in a 7-mile or an 8-mile wind. No. 21 is doing the least work of the five, and requires about an 11-mile wind to start it. It is a wooden mill working direct stroke, while the others are steel mills and back-geared.

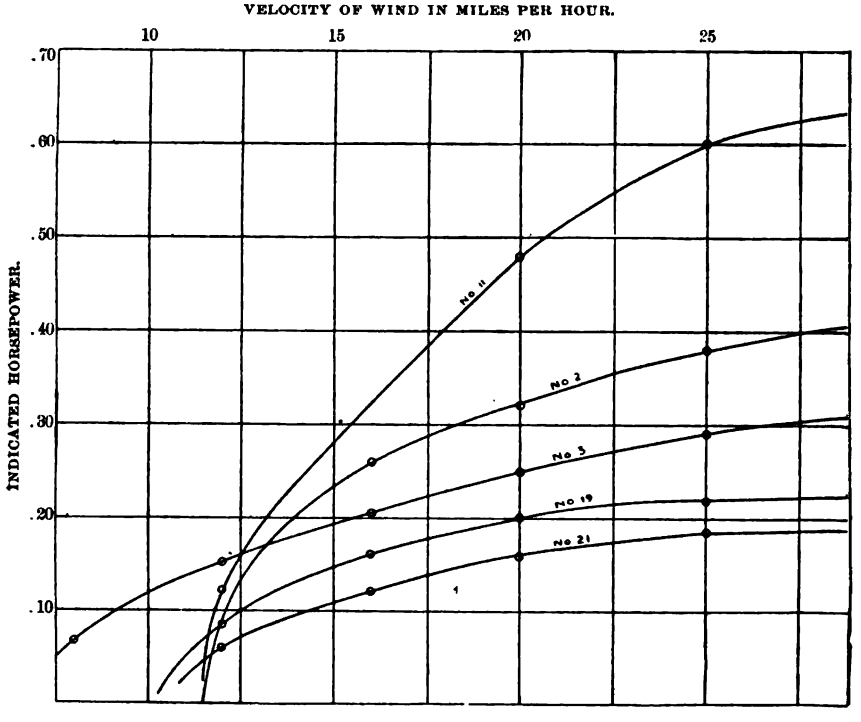


FIG. 30.—Diagram showing relation between horsepower and wind velocity for five 12-foot mills.

It must be understood that in this comparison no correction or allowance is made for the difference in temperature and barometric pressure, nor for the fact that in the case of Nos. 11 and 19 the pumps are on well points, while in the others they are in open wells.

It will be seen that none of the curves in fig. 30 reach a maximum below 30 miles an hour. They do, however, for some higher velocities, since the work per stroke of pump is nearly constant for each pump for all velocities, though not the same for one pump as for another. The curves also give the relation between wind velocity and the number of strokes of pump per minute.

In fig. 31 the curve for No. 18 is seen to reach a maximum at about 25 miles an hour. The others reach their maximum points at veloci-

ties of about 30 miles an hour. These maximum points are points of greatest speed, and are produced by a reduction of wind area, the wind wheel turning out of the wind. This make of mill is seen to "govern," or turn out of the wind, at a lower velocity than other makes.

Comparing the curves in fig. 31, we see that the pump doing the most work at high wind velocities is No. 5, which is also the one most heavily loaded. The principal differences between Nos. 4 and 18, the pumps doing the most and the least work for velocities less than 22 miles an hour, are in the load and the well. No. 4 has five-ninths of the load of No. 18, and is on a well point. The two pumps doing the least work are on well points.

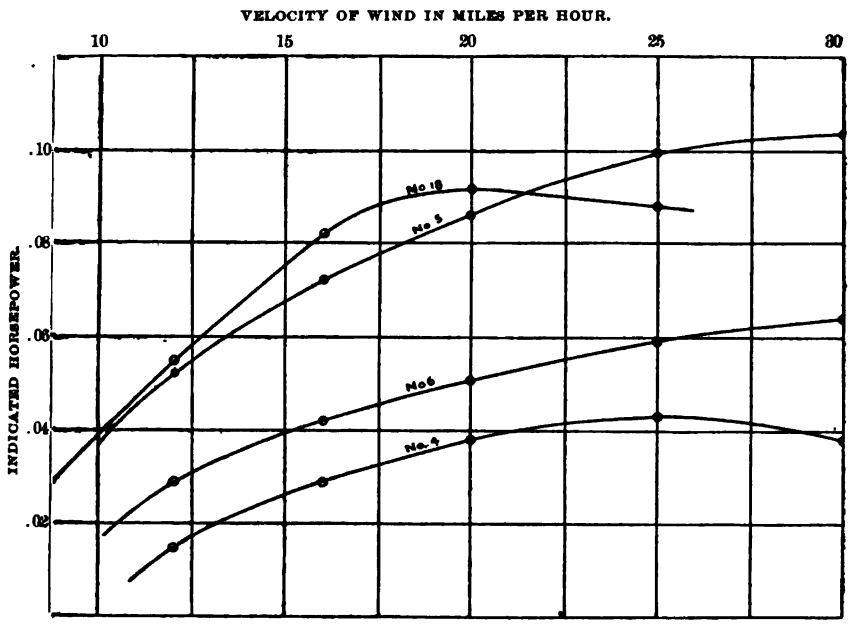


FIG. 31.—Diagram showing relation between horsepower and wind velocity for four 8-foot mills.

Mill No. 25 is used to pump water for stock. Comparing it with, say, No. 5, we find that its load is about five-ninths as great, and that it is doing about one-tenth as much work as the latter. It will start with a wind velocity of about 6 miles an hour, while the latter requires a wind velocity of about 8.5 miles an hour.

It is interesting to compare the results of Nos. 35 and 36. The former is an 8-foot back-geared steel mill, heavily loaded; the latter is a 22.5-foot direct-stroke mill, lightly loaded. The wind exposure of both is very good. The discharge per stroke of No. 35 is 1.1 quarts, that of No. 36 about 3 quarts. The lift of the former is 58 feet, the lift of the latter 39 feet. The load per stroke of the former is 133 foot-pounds, that of the latter 248 foot-pounds. No. 35 starts in about a

9-mile wind, No. 36 in a 6-mile wind. In an 8-mile wind the former is doing no work, the latter is making 4.8 strokes per minute; but in a 12-mile wind the former is making 19 strokes per minute, the latter only 12; for all higher velocities the former is making 60 per cent more strokes than the latter. For wind velocities above 12 miles an hour the horsepower of the 22.5-foot mill is only from 7 to 17 per cent greater than that of the 8-foot mill. The difference is really not so great as this, as the actual discharge is not so great as we have supposed.

PRESSURE-TANK SYSTEM.

Mill No. 37 is of special interest, in that the water is forced into a pressure tank instead of into an elevated tank. This system is said to be in use to some extent in the Eastern States, and is just coming into use in the West. The advantage claimed for it is that the tank can be placed in a cellar or under ground, where it will not freeze, instead of on an elevated structure. A hydraulic regulator is used, which causes the mill to turn out of the wind when the pressure in the tank reaches a certain amount. In this case the tank, which is located in a cellar, is 2 feet in diameter and 9.5 feet long; it is about 170 feet from the well and about 43 feet above the surface of the water in the well. The discharge (1.05 quarts per stroke) is pumped into the tank, then forced by the confined air through a 1-inch iron pipe to the hydrants, then through a line of rubber hose to the desired point. The pressure in the tank diminishes with the reduction in the quantity of water in it. For different volumes of air it is as follows:

Pressures in tank for different volumes of air.

Air volume.	Pressure.
Barrels.	*Pounds.*
11.1	0
5.6	14.7
3.7	29.4
2.8	44.1
2.2	58.8
1.9	73.5
1.6	88.2
1.4	103.0

The objection to this system can easily be seen from this table. When the pressure is 103 pounds per square inch the withdrawal of 0.2 barrel of water from the tank lowers the pressure to 88.2 pounds, and the withdrawal of an additional 0.3 barrel lowers the pressure to 73.5 pounds; in other words, the withdrawal of a half barrel of water when the pressure is 103 pounds lowers the pressure 30 pounds. We found, on trial, that when the pump was not working (no wind), and the pressure was at 59 pounds, one hose stream through a $\frac{3}{16}$-inch nozzle reduced the pressure to 50 pounds in one minute and

to 45 pounds in two minutes. In seven minutes it fell to 30 pounds, and in seventeen minutes from the time of opening the cock it fell to 20 pounds. In a 20-mile to 30-mile wind, and with one hose stream running, the pump kept the pressure in the tank at about 30 pounds. Where only a small amount of water is needed at a time this system will probably give satisfaction. By the use of a larger tank more water can be obtained for a given reduction of pressure.

It will be noticed that this is the same size and make of mill as No. 2, but that it is altogether different from No. 41. No. 2 is more heavily loaded than No. 37. The heaviest load on No. 37 is 472 foot-pounds per stroke of pump; the heaviest load on No. 2 is 536 foot-pounds per stroke. No. 2 is doing the most useful work, but the hydraulic regulator acts on No. 37 to turn it partly out of the wind for the load 43 plus 75 pounds. No. 41 is doing very little work, but it is lightly loaded and working direct stroke.

COMPARISON OF THREE PUMPING AERMOTORS.

Comparing the 16-foot Aermotor No. 9, the 12-foot Aermotor No. 3, and the 8-foot Aermotor No. 5, a diagram was plotted of the horsepower of these three mills, drawn to the same scale. They were found to start at about the same wind velocity, viz, 7 miles an hour, which indicates about the same total load. In a 25-mile wind the 8-foot mill was found to be yielding 0.1 horsepower, the 12-foot mill 0.29 horsepower, and the 16-foot mill 0.6 horsepower. In other words, the 12-foot mill was doing about three times and the 16-foot mill about six times more work than the 8-foot mill. The conclusion can not be drawn from this that the powers of the mills are to each other in these ratios, since the pump efficiencies are not the same.

USEFUL WORK OF TWO PUMPING MILLS IN A GIVEN TIME.

The useful work which two pumping mills of the same wind area, exposure, pump efficiency, and general character will do depends on the load on the mill and the wind velocity. If the mill is heavily loaded it will do more work at wind velocities of 12 or more miles an hour and less work at lower velocities than one of lighter load. The useful work done in a given time is the product of the work done per hour at the mean velocity multiplied by the number of hours. If the mean velocity at a given place is low, the mill load must be less for maximum work than that at a place where the mean velocity is higher. To illustrate this fact, we will use the results of the tests of two 12-foot pumping mills—No. 11, heavily loaded and giving a greater horsepower at high wind velocities than any other mill tested, and No. 3, giving the greatest power at low velocities. The useful work per stroke of pump is 844 foot-pounds for No. 11 and 415 foot-pounds for No. 3. The useless work of the former is greater than that of the latter, since the pump of the former is on

two well points, while the pump of the latter is in an open well. The relation between the horsepower and the wind velocity is shown in fig. 30. The curves are seen to cross each other at a wind velocity of 12.5 miles an hour. For less velocities than that No. 3 is doing more work per hour than No. 11, and for greater velocities No. 11 is doing more work than No. 3. If the velocity were, say, not more than 13 miles an hour, it is very evident that mill No. 3 would do more work in a given time than mill No. 11.

There is no record of wind movement at Garden, Kansas (where most of these tests of pumping mills were made), for any considerable length of time. There is one, however, for Dodge, 50 miles east of Garden, kept by the United States Weather Bureau, which may be used for this purpose. The following table gives the number of hours per month for the six months April to September, for the years 1889 to 1895, inclusive, when the wind movement was, respectively, 0 to 5, 6 to 10, 11 to 15, 16 to 20, 21 to 25, 26 to 30, 31 and more miles per hour.

Mean wind movement at Dodge, Kansas, for the seven years 1889 to 1895.

Month.	0-5 miles.	6-10 miles.	11-15 miles.	16-20 miles.	21-25 miles.	26-30 miles.	31 and greater.
	Hours.	*Hours.*	*Hours.*	*Hours.*	*Hours.*	*Hours.*	*Hours.*
April	116	175	157	113	76	43	40
May	116	195	168	120	74	39	32
June	120	187	189	111	86	49	28
July	144	218	176	117	57	23	9
August	178	230	152	99	62	18	5
September	166	182	152	98	75	34	18
Mean	140	198	157	109	72	34	22

It will be seen from this table that the wind velocity at Dodge is 5 miles or less per hour for 140 hours per month. During this time neither of these mills (Nos. 3 and 11) will do any work, as neither will start in a 5-mile wind.

The velocity is from 6 to 10 miles an hour for 198 hours a month. Mill No. 3 will start in about a 7-mile wind, and hence will run about four-fifths of this time, or 158 hours. No. 11 requires 11.5 miles of wind to start it, and will do no work during this time.

The velocity is 11 to 15 miles an hour for 157 hours during the month. No. 3 will work all of this time, and No. 11 about nine-tenths of the time, or 141 hours.

Both mills will run at all higher velocities. At Dodge, mill No. 3 will run (if in the wind) about 75 per cent of the time, and No. 11 about 51 per cent of the time. For convenience, these results have been tabulated.

Comparative results of tests of two pumping mills.

Wind velocities, miles per hour.	Mill No. 3.			Mill No. 11.		
	Hours per month.	Horse-power.	Product.	Hours per month.	Horse-power.	Product.
6 to 10	158	0.067	10.6	0	0.00	0.0
11 to 15	157	0.168	26.4	141	0.19	26.8
16 to 20	109	0.230	25.1	109	0.40	43.6
21 to 25	72	0.277	19.9	72	0.56	40.3
26 to 30	34	0.308	10.5	34	0.63	21.4
31 and greater	22	0.320	7.0	22	0.64	14.1
Total			99.5			146.2

The second and fifth columns in this table give the number of hours during the mean month that each mill will run with a wind velocity of from 6 to 10 miles an hour. The third and sixth columns give the horsepower for the mean velocity; for example, 0.168 is the horsepower for No. 3 at a wind velocity of 13 miles an hour, and 0.19 is the horsepower for No. 11 at the same velocity. The fourth and seventh columns (the product of the number of hours and the horsepower) give the horsepower that each mill will yield during the month. It will be seen that No. 11 is doing 31 per cent more useful work than No. 3. If this comparison be made for the month of August, it will be found that No. 11 will do 26 per cent more useful work during that month than No. 3.

PROPER LOAD.

It will be seen from what has just preceded, that the useful power of a pumping mill depends to a great extent on its load. If the water is needed constantly and there is little or no storage, as in the case of water for stock, the mill must be lightly loaded, and the useful work it will do is small. If, however, there is plenty of storage, the mill will pump the largest amount of water if heavily loaded. If there were some automatic device for increasing the load on the mill as the wind velocity increases, the problem of proper load would be solved. But such a device seems difficult to construct.

The following table gives data for back-geared steel irrigating mill and good pumps for use in the semiarid regions of the West, especially for Kansas and Nebraska.

Data regarding mills and pumps for use in semiarid regions.

Diameter of mill.	Load per stroke.	Quantity delivered per stroke.	Lift.	Cylinder capacity.	Starting wind velocity (per hour).	Speed per minute in 20-mile wind.	Discharge per hour.	Discharge per 24 hours.
Feet.	*Ft.-lbs.*	*Quarts.*	*Feet.*	*Inches.*	*Miles.*	*Strokes.*	*Gallons.*	*Acre-ft.*
8	125	2.0	30	4.5× 8	9	27	810	0.059
12	600	9.6	30	8×12	8 to 9	20	2,880	0.212
16	1,100	17.6	30	9×16	7 to 8	16	4,224	0.311

For half this lift, or 15 feet, the cylinder capacity should be nearly doubled, and for double the lift the cylinder capacity should be about half that given in the table.

In Part II of this paper, published as Water-Supply and Irrigation Paper No. 42, will be found a discussion of the writer's experiments with power mills, a comparison of pumping mills with power mills, a discussion of the effect of tension of spring on the horsepower of mills, a mathematical discussion of the tests of two Aermotors, a discussion of the action of air on the sail of an Aermotor, a discussion of the useful work of two power mills in a given time, discussions of the results of tests of a Jumbo mill and of a Little Giant mill, a comparison of the Little Giant and Jumbo mills, a comparison of the Little Giant mill and the 8-foot Aermotor, a discussion of the indicated and true velocities of windmills, a comparison of the writer's experiments with those of other experimenters, and economic considerations.

[For index see end of Part II, Water-Supply Paper No. 42.]

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56TH CONGRESS, 2d Session. } HOUSE OF REPRESENTATIVES. { DOCUMENT No. 485.

DEPARTMENT OF THE INTERIOR

WATER-SUPPLY

AND

IRRIGATION PAPERS

OF THE

UNITED STATES GEOLOGICAL SURVEY

No. 42

THE WINDMILL: ITS EFFICIENCY AND ECONOMIC USE, PART II.—MURPHY

WASHINGTON
GOVERNMENT PRINTING OFFICE
1901

UNITED STATES GEOLOGICAL SURVEY

CHARLES D. WALCOTT, DIRECTOR

THE WINDMILL:

ITS EFFICIENCY AND ECONOMIC USE

PART II

By EDWARD CHARLES MURPHY

WASHINGTON
GOVERNMENT PRINTING OFFICE
1901

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LETTER OF TRANSMITTAL.

DEPARTMENT OF THE INTERIOR,
UNITED STATES GEOLOGICAL SURVEY,
DIVISION OF HYDROGRAPHY,
Washington, January 14, 1901.

SIR: I have the honor to transmit herewith a manuscript entitled The Windmill: Its Efficiency and Economic Use, Part II, by Edward Charles Murphy, formerly professor of civil engineering in the University of Kansas, and to request that it be published in the series of Water-Supply and Irrigation Papers.

This manuscript is a continuation of the material published as Paper No. 41, it being necessary to divide the manuscript on account of the limitation imposed by law of 100 printed pages for each pamphlet.

Very respectfully,

F. H. NEWELL,
Hydrographer in Charge.

Hon. CHARLES D. WALCOTT,
Director United States Geological Survey.

THE WINDMILL: ITS EFFICIENCY AND ECONOMIC USE.

PART II.

By EDWARD CHARLES MURPHY.

EXPERIMENTS BY WRITER—CONTINUED.

In Part I of this paper, published as Water-Supply and Irrigation Paper No. 41, will be found a classification of windmills, a discussion of regulating devices, a synopsis of early experiments with windmills, and a discussion of the writer's experiments with pumping mills. This part (II) of the paper contains the results of the writer's experiments with power mills, a comparison of pumping mills with power mills, and discussions of various facts developed by the tests, together with a comparison of the writer's experiments with those of other experimenters, and the economic considerations of the subject.

POWER MILLS.

The power mill differs essentially from the pumping mill in that the latter gives a reciprocating motion to a pump piston, while the former gives a rotary motion to a vertical shaft, and this, in turn, to a horizontal shaft, which drives the grinder or other machine. The mechanism by which this is accomplished in the Aermotor is shown in figs. 33 and 34. The small plane cogwheel makes three revolutions to one revolution of the wind wheel, and the small beveled cogwheel makes two revolutions to one revolution of the small plane wheel; so that the vertical shaft makes six revolutions to one revolution of the wind wheel; or, as we say, the shaft is geared forward 6 to 1. The two beveled cogwheels of the foot gear (fig. 34) change the motion around a vertical axis to a motion around a horizontal axis without changing the rate of speed. In fig. 7, Part I, which shows the pumping mill, it will be seen that the large cogwheel which gives the up-and-down motion to the piston makes one revolution to each 3.3 revolutions of the wind wheel, or that the pump is geared back 3.3 to 1; so that the vertical shaft of a power mill makes twenty revolutions to one stroke of a pump worked by a pumping mill the wind wheel of which is running at the same rate as that of the power mill. The

term "geared mill" is sometimes applied to power mills, but inappropriately, since the pumping mill also is geared. The latter is geared back, the former is geared forward.

Power mills are heavier than pumping mills. They ordinarily do more work and carry heavier loads than the latter. The load on the pumping mill is constant for all wind velocities, but it may be varied in the power mill. The grinder is made so that as the speed increases the quantity of corn which enters increases, and thus the load and work done are increased. The mill is expected to do three or four kinds of work—for example, pump water, shell and grind corn, and turn a grindstone. In a light wind the pump only can be worked, but as the wind increases one after another of the three other machines can be set at work, and thus the load be suited to the velocity of the wind and the mill be made to do the maximum amount of work. Power mills are not made smaller than 12 feet in diameter, for the reason that a small size will not give power enough to be of account except for pumping. The ordinary steel power mills are 12 feet, 14 feet, and 16 feet in diameter.

The power that a windmill is capable of developing can be determined better from a power mill than from a pumping mill, because the efficiency of the pump—which may be anywhere from 20 to 85 per cent—is eliminated, and because the load on the mill can be varied at will, and thus the effect of the load on the power of the mill be determined for different wind velocities.

METHOD OF TESTING.

The power was measured by the use of a Prony friction brake placed on an iron pulley on the foot gear or horizontal shaft. The brake was of wood, and had an arm 3 or 4 feet long. Near the end of this arm was fastened a spring balance reading to quarters of a pound. By turning the nuts on the brake the spring balance could be made to read any desired amount. As the brake on the pulley was tightened, the reading of the spring balance was increased and the number of revolutions of the shaft decreased. The brake is shown in Pl. XV. The speed of the shaft was found in one of three ways—whichever was most convenient. A small electric device was used whenever it could conveniently be attached to the wind wheel. The clicks of this instrument could easily be counted, and gave the number of revolutions of the wind wheel for each half mile of wind movement. A speed counter was used, but did not prove satisfactory. Whenever the electric device could not conveniently be employed, the number of revolutions of the wind wheel was found by counting the revolutions of a mark on the wind wheel as reflected in a mirror conveniently placed.

To illustrate: If u is the number of revolutions per minute of the brake pulley as found from the revolutions of the wind wheel per

half mile of wind, L the load in pounds as read from the spring balance, and R the length of the arm, then the useful work, in foot-pounds per minute, is—$W = 2\pi R u L$, and the horsepower is—H. P. $= 2\pi R u L \div 33{,}000$.

The number of revolutions per minute of wind wheel was found for each mill for from two to six different brake loads, for wind velocities as small as would keep the mill working for the particular load used

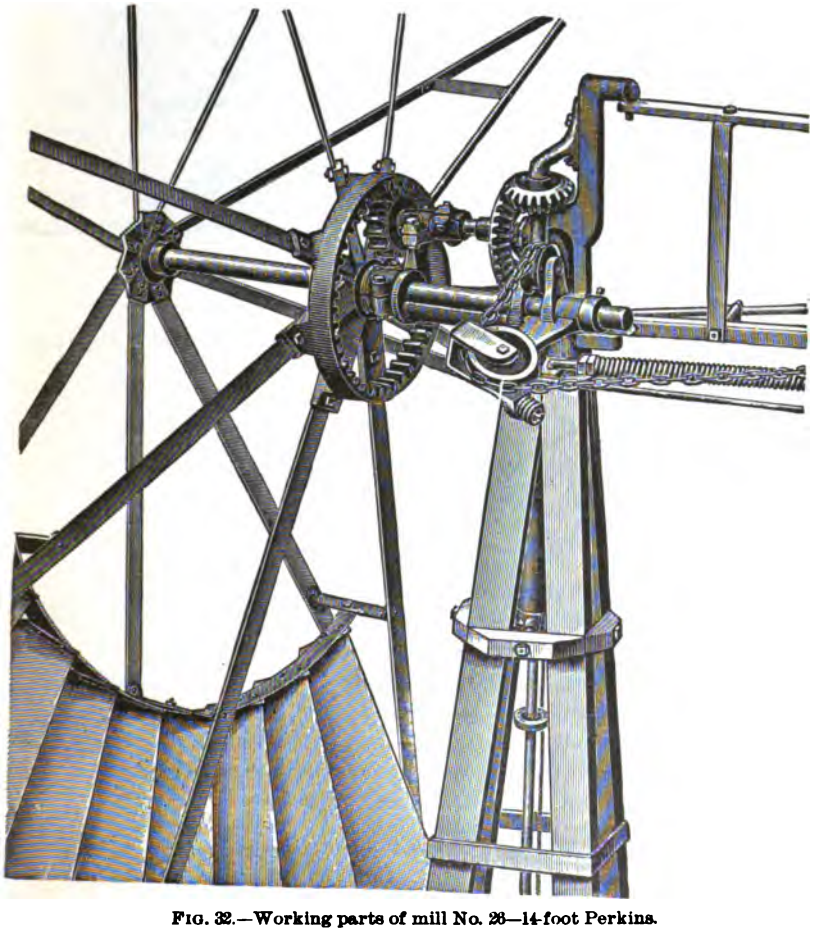

FIG. 32.—Working parts of mill No. 26—14-foot Perkins.

up to about 25 miles an hour. That is, the reading of the spring balance was kept as nearly constant as possible until we had obtained points and a curve like that shown in fig. 10. Then the load was changed and the tests continued in the same way, getting another curve. From these curves the number of revolutions of wind wheel per minute for different loads and wind velocities was easily found, as before indicated. These are given in the table of results for each mill tested, and in many cases are also shown by diagram. The horse-

power of any mill for different loads and velocities is easily found by the foregoing formula. These are given in the tables of results of tests, and are also shown by diagrams.

MILLS TESTED.

Mill No. 26.—This is a 14-foot Perkins steel power mill on a 40-foot steel tower, made by the Perkins Windmill Company, of Mishawaka, Indiana. The working parts are shown in fig. 32. The wind wheel has 32 curved sails, each 41 by 14.25 by 7.75 inches, set at an angle of 31° with the plane of the wheel. The shaft is geared forward 6 to 1. The radius of the brake pulley was 5 inches, the length of brake arm 33.5 inches. This mill was tested twice. Between the dates of testing some repairs were made to the shafting, causing the cogwheels to bind less tightly. The following figures are those obtained from the second test. The mill had been in use only about one year, and showed very poor workmanship. The results of the test are as follows:

Results of test of mill No. 26—14-foot steel Perkins.

Load on brake.	Load per revolution of wind wheel.	Number of revolutions of wind wheel per minute at given wind velocities (per hour).					Useful horsepower at given wind velocities (per hour).				
		8 miles.	12 miles.	16 miles.	20 miles.	25 miles.	8 miles.	12 miles.	16 miles.	20 miles.	25 miles.
Pounds.	*Ft.-lbs.*										
6	645	.....	.....	16	31	48	.....	.....	0.313	0.609	0.937

Mill No. 27.—This is a 12-foot Aermotor on a 30-foot steel tower. The wind wheel is like that of mill No. 3 (see pp. 29 to 30, Part I). The horizontal shaft is geared forward 6 to 1. Fig. 33 shows the working parts, and fig. 34 the foot gear. The brake pully was 9.5 inches in diameter and was fastened to the foot gear at *a* in fig. 34. The brake arm was 35.25 inches in length. The mean temperature during the test was 46° F., and the mean barometric pressure 28.9 inches. The results of the tests are as follows:

Results of tests of mill No. 27—12 foot Aermotor.

Load on brake.	Load per revolution of wind wheel.	Number of revolutions of wind wheel per minute at given wind velocities (per hour).					Useful horsepower at given wind velocities (per hour).				
		8 miles.	12 miles.	16 miles.	20 miles.	25 miles.	8 miles.	12 miles.	16 miles.	20 miles.	25 miles.
Pounds.	*Ft.-lbs.*										
0	0	30	49	63	75	87	.....	.....	.....	.....	.....
2	222	16	43	57	70	81	0.089	0.285	0.386	0.458	0.523
4	444	.....	23	48	65	77	.....	0.303	0.653	0.890	1.02
6	666	.....	.....	12	50	72	.....	.....	.....	1.03	1.45

The revolutions of wind wheel per minute for the four brake loads 0, 2, 4, and 6 pounds, respectively, are shown in fig. 35. The pull necessary to overcome the frictional resistance was found by standing on the platform of the mill and slowly turning the wind wheel around with a spring balance. This was checked by winding a

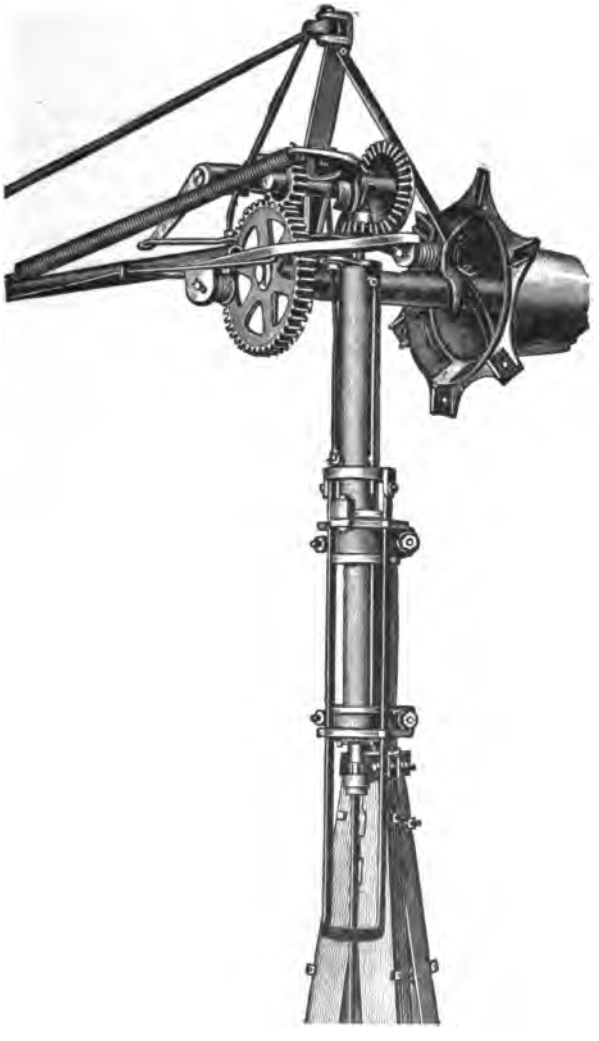

FIG. 33.—Working parts of mill No. 27—12-foot Aermotor.

cord around the circumference of the wind wheel, and, standing on the ground, moving the wheel when there was no wind by pulling on the spring balance attached to the cord. A pull of 1.25 pounds applied at the circumference was sufficient to overcome this resistance at a low velocity. The work done in overcoming this resistance is $1.25 \times 2\pi \times 6 = 47.1$ foot-pounds per revolution. The work done per

revolution of wind wheel per pound on the brake arm is—$2\pi \times 35.25 \times 6 \div 12 = 111$ foot-pounds. The ratio of these is $47.1 \div 111 = 0.425$ pound. Hence a brake load of 0.425 pound is equivalent to the friction load.

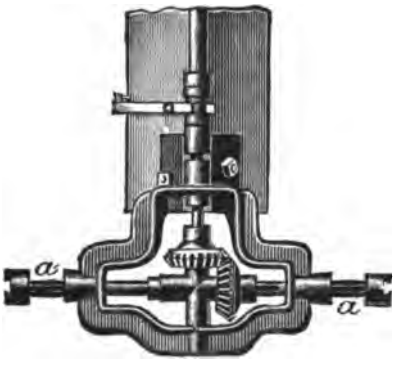

FIG. 34.—Foot gear of mill No. 27—12-foot Aermotor. *a* indicates point where brake pulley was attached.

The effect of each additional 2 pounds load on the brake in reducing the speed of the wheel at different wind velocities is clearly shown here. It is seen that an added load makes a greater proportionate reduction in the speed when the velocity is low than when it is high. Thus, in an 8-mile wind the addition of 2 pounds to the load reduces the speed 50 per cent, while in a 25-mile wind the same load reduces the speed only about 7 per cent.

It will be seen that for wind velocities above a certain amount, with the load not too great, each additional pound of load reduces the speed of the wheel by about the same amount. For example, in a 25-mile wind the addition of 2 pounds changes the speed from 87 revolutions to 81 revolutions. The addition of 2 pounds more changes

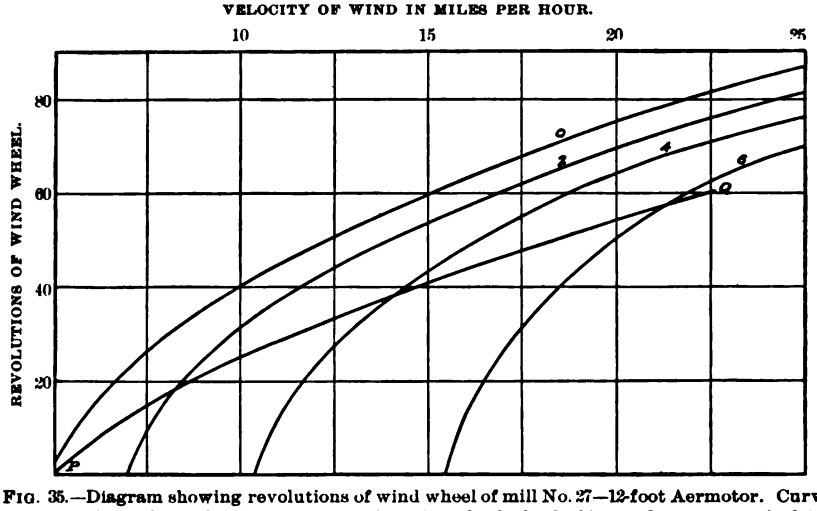

FIG. 35.—Diagram showing revolutions of wind wheel of mill No. 27—12-foot Aermotor. Curve marked *o* is for no brake load; curve marked 2 is for a brake load of 2 pounds; curve marked 4 is for a brake load of 4 pounds; curve marked 6 is for a brake load of 6 pounds; curve *PQ* is speed curve for maximum power.

the speed from 81 revolutions to 77 revolutions. It will be seen that as the load increases the increment of wind velocity necessary to start the mill increases more rapidly than the increment of loading. That is, the 0-pound load curve starts in about a 4.5-mile wind, the 2-pound

curve in about a 7-mile wind, the 4-pound curve in about a 10.5-mile wind, and the 6-pound curve in about a 15.5-mile wind. The difference between these starting velocities is constantly increasing. A diagram was platted showing the horsepower of this mill for the 2-pound, 4-pound, and 6-pound brake loads. The curves showed that for any brake load the power of the mill increased rapidly as the wind velocity increased, and that it reached a maximum for some velocity greater than 30 miles an hour. As the load increased the velocity required to start the mill increased rapidly and the curve became steeper. For a given wind velocity the power increased rapidly as the load increased. For a velocity of 25 miles an hour the power was nearly proportional to the load for loads of less than 6 pounds. It showed that when the velocity was less than 12 miles an hour a 4-pound load was too great, and when it was less than 19 miles an hour a 6-pound load was too great. It showed also that the efficiency decreased as the wind velocity for a given load increased, and that it increased as the load increased. The efficiency for a load of 2 pounds and a wind velocity of 9 miles an hour was 40 per cent. At 14 miles an hour and with a 4-pound load it was 36 per cent. If the load at that velocity was reduced to 2 pounds, the efficiency was reduced to 24 per cent. Finding the efficiency by using the wind area (area of circle 12 feet in diameter) instead of the sail area, as is sometimes done, the foregoing efficiency of 40 per cent with a 2-pound load in a 9-mile wind became 26 per cent.

The results for this mill will be discussed from a mathematical point of view further on.

Mill No. 28.—This is a 16-foot Althouse wooden power mill manufactured by Althouse, Wheeler & Company, of Waupun, Wisconsin. It is shown in fig. 36. The axis of the wind wheel is 32 feet above the ground and 15 feet above the roof of a near-by blacksmith's shop. The wind wheel has 130 sails, each 48 by 4 by 1.5 inches, set at an angle of 32° to the plane of the wheel. Two half sails are missing and two others are slightly injured, making a loss of about one and a half sails. The horizontal shaft, which works a sheller, grinder, emery wheel, and wood saw, is geared forward 8.377 to 1. A pull of from 7 to 13 pounds at a distance of 6 feet from the center was necessary to start the mill, showing it to be a hard-running one. The brake pulley was 8 inches in diameter, the brake arm 3.5 feet long. A second visit to this mill was necessary in order to get results for high velocities. During the interval between the tests a 510-pound fly wheel was put on the shaft, which steadied the motion of the mill somewhat. The owner is well pleased with the action of this balance wheel. Single measurements of the power of this mill for the same load and wind velocity differ considerably. It is very evident that this mill is not high enough; if it were 30 or 40 feet higher it would

give better results. It is with the aid of better results obtained from tests of other mills of similar make that we are able to give the results in the following table and in the diagrams.

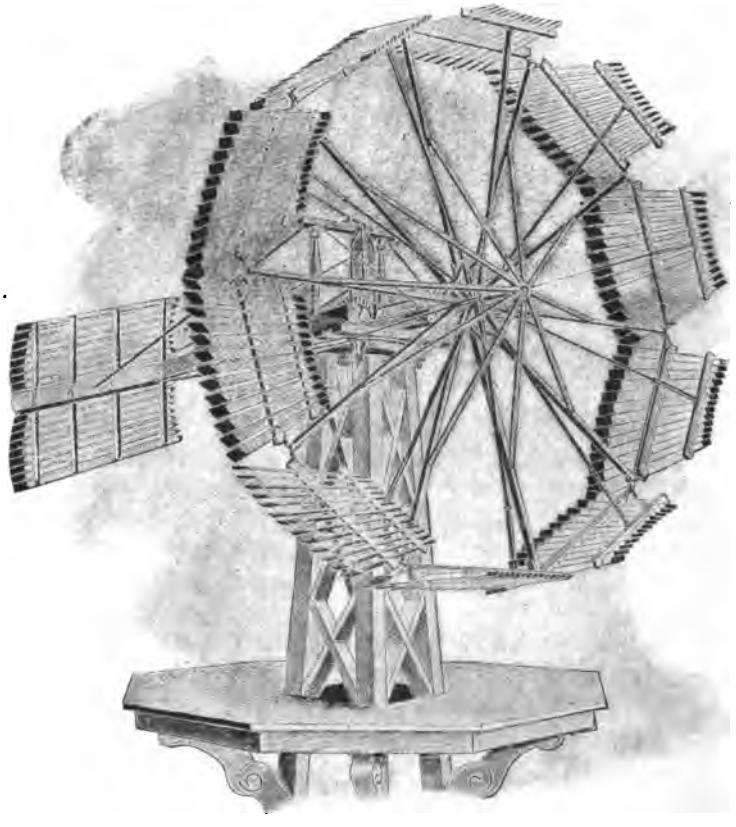

FIG. 36.—View of mill No. 28—16-foot wooden Althouse.

Results of tests of mill No. 28—16-foot wooden Althouse.

Load on brake.	Load per revolution of wind wheel.	Number of revolutions of wind wheel per minute at given wind velocities (per hour).					Horsepower of mill at given wind velocities (per hour).				
		8 miles.	12 miles.	16 miles.	20 miles.	25 miles.	8 miles.	12 miles.	16 miles.	20 miles.	25 miles.
Pounds.	*Ft.-lbs.*										
0	0	13	23	30	36	40	.....	.....	.....	.....	.....
1¾	427	.....	20	27	31	36	.....	0.26	0.35	0.40	0.46
G.	.....	.....	5	13	20	28	.....	.....	.....	.....	.....
5¾	914	.....	.....	19	23	27	.....	.....	0.52	0.64	0.75
8¾	1,462	.....	.....	8	13	15	.....	.....	0.35	0.60	0.67

Fig. 37 shows the number of revolutions per minute of the wind wheel of this mill for the brake loads 0, 1.75, 5.75, and 8.75 pounds,

and for grinder load. The curve for the grinder load is a nearly straight line. This is due to the fact that the grinder is constructed so that as its speed increases the amount of corn it receives increases; thus the load increases automatically as the wind velocity increases. By comparing these speed curves, as they may be called, with those of fig. 35, for the Aermotor, it will be seen that the speed of the latter is much greater than that of mill No. 28.

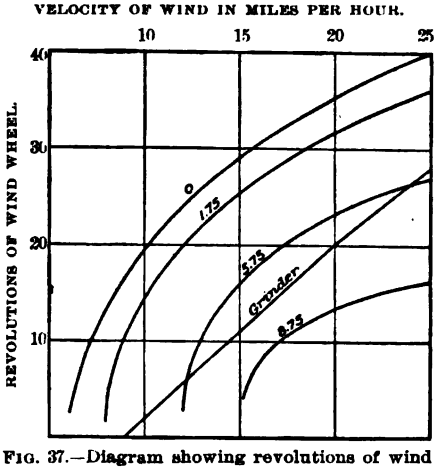

FIG. 37.—Diagram showing revolutions of wind wheel of mill No. 28—16-foot wooden Althouse. Curves marked 0, 1.75, 5.75, 8.75, and grinder are for brake loads of 0, 1.75, 5.75, and 8.75 pounds, respectively, and for grinder load.

Fig. 38 shows the horsepower of this mill for three brake loads—1.75 pounds, 5.75 pounds, and 8.75 pounds. The latter load is too great for the mill. By comparing the results for this mill with those for the 12-foot Aermotor (fig. 35) it will be seen that the latter mill is superior to the 16-foot wooden mill.

Mill No. 29.—This is a 16-foot Aermotor like that shown in Pl. XV. It is manufactured by the Aermotor Company, of Chicago, Illinois. The tower is of wood, 42 feet to the axis of the wheel. The wind wheel has 18 curved sails, each 59 by 25.75 by 10.5 inches, set at an angle of 30° to the plane of the wheel. It is used for shelling and grinding corn and for working a small pump. The shafting is 20 feet above the ground, near the roof of a granary. It is geared forward 6 to 1, and arranged so that the pump makes 1.011 strokes to each revolution of the wind wheel. The pump lifts 0.022 gallon per stroke a distance of about 40 feet. It has a cylinder 1.5 inches in diameter and a stroke of 8 inches. The supply pipe is on a well point. The brake pulley is 12 inches in diameter, the brake arm 3.75 feet long. The test was continued until the shafting failed. The mean baro-

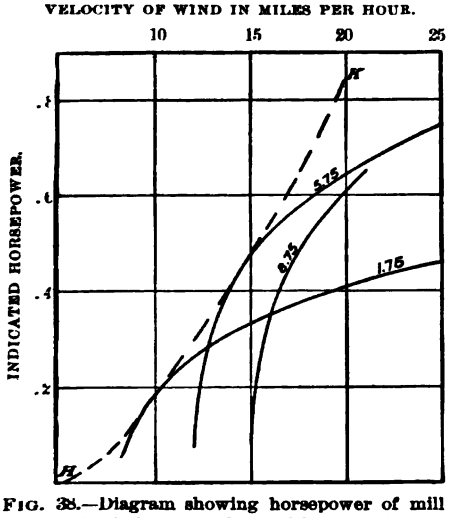

FIG. 38.—Diagram showing horsepower of mill No. 28—16-foot wooden Althouse. Curves marked 1.75, 5.75, and 8.75 show horsepower for brake loads of 1.75, 5.75, and 8.75 pounds, respectively; dotted curve *HK* shows maximum power.

metric pressure was 27.8 inches, the mean temperature 70° F. This mill had been in nearly constant use about five years. It replaced a 22.5-foot Halliday. The owner claims that the 16-foot steel mill does more work than the 22.5-foot Halliday wooden mill did. The results of the tests are as follows:

Results of tests of mill No. 29—16-foot Aermotor.

Load on brake.	Load per revolution of wind wheel.	Number of revolutions of wind wheel per minute at given wind velocities (per hour).					Horsepower of mill at given wind velocities (per hour).				
		8 miles.	12 miles.	16 miles.	20 miles.	25 miles.	8 miles.	12 miles.	16 miles.	20 miles.	25 miles.
Pounds.	*Ft.-lbs.*										
1¼	212	20	35	.....	.....	.....	0.13	0.23	.....	.....	.....
P+1¼	487	11	30	41	.....	.....	0.17	0.44	0.636	.....	.....
P+2¼	629	.....	25	35	.....	.....	.....	0.48	0.714	.....	.....

Fig. 39 shows the number of revolutions of the wind wheel for three loads—212 foot-pounds, 487 foot-pounds, and 629 foot-pounds. Although these results are incomplete, on account of the failure of the shafting, they are complete up to a wind velocity of 15 miles an hour, and when studied in connection with the complete test of a mill of the same size and make (No. 44) it will be seen that this mill has about the same power for the same loads at any given wind velocity.

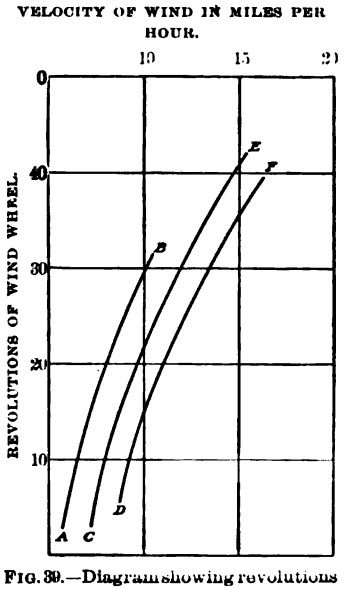

FIG. 39.—Diagram showing revolutions of wind wheel of mill No. 29—16-foot Aermotor. Curve *AB* is for a load of 212 foot-pounds; *CE* is for a load of 487 foot-pounds; *DF* is for a load of 629 foot-pounds.

Mill No. 30.—This is a 16-foot wooden power mill known as an Irrigator, used for lifting water. (For description see pp. 49–50, Part I.) Two brake loads (2 pounds and 16 pounds) were used on an arm 2.5 feet long. Curves showing the number of revolutions of the wind wheel per minute for these loads and the useful elevator load are reproduced in fig. 22, Part I; the horsepowers for these loads are shown in fig. 23, Part I.

Mill No. 31.—This is a 14-foot Elgin wooden power mill used to lift water with a rotary (Wonder) pump. It is described on pages 50 to 51, Part I.

Mill No. 34.—This is a 14-foot Junior Ideal steel power mill manufactured by the Stover Manufacturing Company, of Freeport, Illinois. The tower is of wood, 41 feet to the axis of the wheel. The wheel has

24 curved sails in eight sections, regulated on the centrifugal principle. Each sail is 49 by 18 by 8 inches, set at an angle of 29° to the plane of the wheel. This is a sectional vaneless mill. In place of a vane there is a counterpoise. It is geared forward 8 to 1. The mill is used for shelling and grinding corn and elevating. The brake pulley is on a line shaft 15 or 20 feet long. In a 12-mile wind the mill ground 12 pounds quite fine for a mile of wind, or at the rate of 144 pounds an hour. In an 18-mile wind it ground 26 pounds for a mile of wind, or at the rate of 468 pounds an hour. The grinder was made by the Baker Manufacturing Company, of Evansville, Wisconsin. The wind was unsteady, the temperature high—100° in the shade at noon. The results of the tests are as follows:

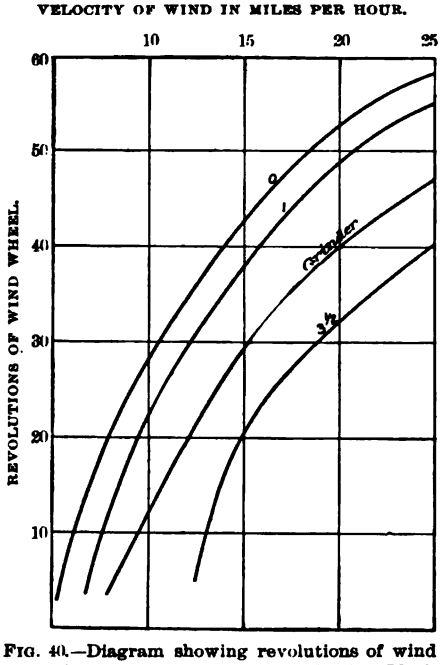

FIG. 40.—Diagram showing revolutions of wind wheel of mill No. 34—14-foot Junior Ideal. Curves marked 0, 1, 3½, and grinder are for brake loads of 0, 1, and 3.5 pounds, respectively, and for grinder load.

Results of tests of mill No. 34—14-foot steel Junior Ideal.

Load on brake.	Load per revolution of wind wheel.	Number of revolutions of wind wheel per minute at given wind velocities (per hour).					Horsepower of mill at given wind velocities (per hour).				
		8 miles.	12 miles.	16 miles.	20 miles.	25 miles.	8 miles.	12 miles.	16 miles.	20 miles.	25 miles.
Pounds.	*Ft.-lbs.*										
0	0	20	34	44	53	58	.....	.....	.....	.....	.....
1	180	13	29	40	49	55	0.07	0.16	0.22	0.27	0.33
G.	G.	4	19	32	39	47	.....	.....	.....	.....	.....
3½	631	0	0	24	32	40	0	0	0.46	0.61	0.76

Fig. 40 shows the number of revolutions of the wind wheel per minute for brake loads of 0, 1, and 3.5 pounds, and for the grinder load. Fig. 41 shows the horsepower for brake loads of 1 and 3.5 pounds.

Mill No. 44.—This is a 16-foot Aermotor on a 40-foot steel tower. (See Pl. XV.) The working parts of the mill are like those shown in fig. 33; the foot gear is like that shown in fig. 34. The sail area is the same as that of mill No. 29, page 91. The power was measured with a wooden brake having an arm 4.67 feet long, on a 10-inch iron pulley on the foot gear. Five brake loads were used—0, 3, 5, 8, and 11 pounds, respectively. The shafting is geared forward 6 to 1.

The results of the tests are as follows:

Results of tests of mill No. 44—16-foot Aermotor.

Load on brake.	Load per revolution of wind wheel.	Number of revolutions of wind wheel per minute at given wind velocities (per hour).					Horsepower of mill at given wind velocities (per hour).				
		8 miles.	12 miles.	16 miles.	20 miles.	25 miles.	8 miles.	12 miles.	16 miles.	20 miles.	25 miles.
Pounds.	*Ft.-lbs.*										
0	0	23	38	48.0	56	64.5	.....	.....	.....	.....	.....
3	528	.....	28	41.0	50	58.5	.....	0.45	0.66	0.80	0.94
5	880	.....	13	33.5	44	53.5	.....	0.35	0.89	1.16	1.43
8	1,408	.....	.....	16.0	36	47.0	.....	.....	0.68	1.53	2.01
11	1,936	.....	.....	.....	25	39.5	.....	.....	.....	1.47	2.31

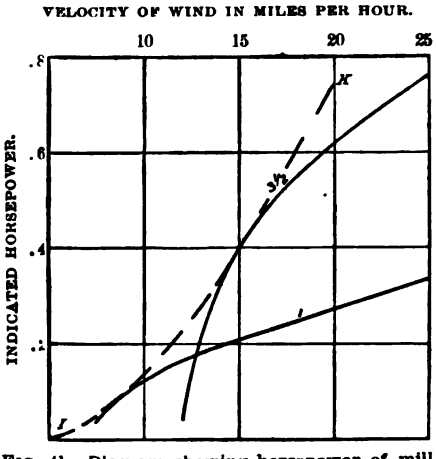

FIG. 41.—Diagram showing horsepower of mill No. 34—14-foot Junior Ideal. Curves marked 3½ and 1 show power for brake loads of 3.5 pounds and 1 pound, respectively; dotted curve *IK* shows maximum power.

Fig. 42 shows the number of revolutions per minute of the wind wheel for these brake loads. The number on each curve indicates the brake load for that curve. These curves are seen to closely resemble the corresponding curves for the 12-foot Aermotor (fig. 35). The 16-foot mill will be seen to start, with no load, in about a 4.5-mile wind—the same as the 12-foot Aermotor. Fig. 43 shows the horsepower of this mill for loads of 3, 5, 8, and 11 pounds, respectively. The curves for this mill closely resemble those of the 12-foot Aermotor. The curves of the latter were platted, but the diagram is not reproduced because of lack of space. It will be shown further on that these load curves are parabolas, and hence that the power increases as the square root of the wind velocity. It will be shown also that the curve of maximum power is a parabola, that the load for it increases nearly as the first power of the wind velocity, and that the speed of the wheel increases also as the first power of the wind velocity.

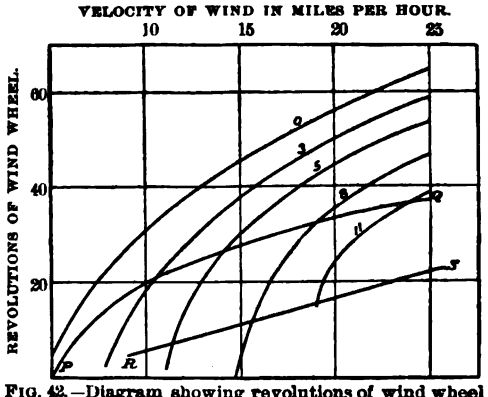

FIG. 42.—Diagram showing revolutions of wind wheel of mill No. 44—16-foot Aermotor. Curves marked 0, 3, 5, 8, and 11 are for brake loads of 0, 3 pounds, 5 pounds, 8 pounds, and 11 pounds, respectively; curve *PQ* is speed of wheel for maximum load; *RS* is load curve for maximum power.

Mill No. 49.—This is a

VIEW OF MILL NO. 44—16-FOOT AERMOTOR.

22.5-foot Halliday wooden power mill on a 43-foot wooden tower. The sail area is in two concentric rings, the outer ring having 144 sails, the inner ring 100 sails, each 43 by 4.5 by 3.5 inches, set at an angle of 25° to the plane of the wheel. The upper gearing has a ratio of 50 to 14, the lower gearing a ratio of 53 to 26, so that the horizontal shaft is geared forward 7.28 to 1. The brake pulley is 8 inches in diameter, the brake arm 4.75 feet long. The mean temperature was 82° F., the mean barometric pressure 28.7 inches. The mill is used for shelling and grinding corn. Four brake loads (0, 1.5, 5, and 9 pounds, respectively) were used, also the grinder load.

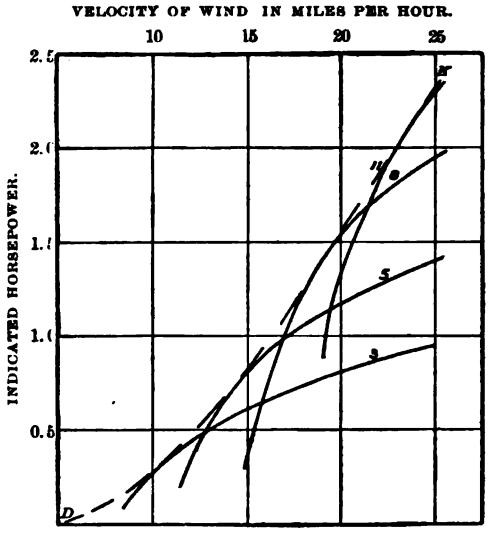

FIG. 43.—Diagram showing horsepower of mill No. 44—16-foot Aermotor. Curves marked 3, 5, 8, and 11 show power for brake loads of 3, 5, 8, and 11 pounds, respectively; dotted curve *DK* shows maximum power.

The results of the tests are as follows:

Results of tests of mill No. 49—22.5-foot wooden Halliday.

Load on brake.	Load per revolution of wind wheel.	Number of revolutions of wind wheel per minute at given wind velocities (per hour).					Horsepower at given wind velocities (per hour).				
		8 miles.	12 miles.	16 miles.	20 miles.	25 miles.	8 miles.	12 miles.	16 miles.	20 miles.	25 miles.
Pounds.	*Ft.-lbs.*										
0	0	11	20	25	30	32	.....	.....	.....	.....	.....
1.5	326	6	16	22	26	30	0.059	0.163	0.217	0.257	0.296
5.0	1,087	.....	11	18	22	26	.....	0.342	0.593	0.724	0.856
G.	.....	.....	8	16	22	.....	.....	.....	.....	.....	.....
9.0	1,956	.....	2	10	15	19	.....	0.118	0.593	0.890	1.126

In a 13.5-mile wind the mill ground 20 pounds of chop quite fine in 4.5 minutes. Fig. 44 shows the number of revolutions per minute of the wind wheel for the five loads. This mill requires a 5.5-mile wind to start it without any load, and it makes only 32 revolutions in a 25-mile wind. This is about half as many as are made under the same conditions by the 16-foot mill No. 44. Since the circumference of the 22.5-foot mill is 1.4 times greater than that of the 16-foot mill, the circumference velocity of the 16-foot mill is 44 per cent greater than that of the 22.5-foot mill. The dotted curve, showing the speed of the wheel for the grinder load, will be seen to be a nearly straight line, showing a

constantly increasing load with increase of wind velocity. Fig. 45 shows the horsepower for three loads, also the maximum horsepower. It will be seen that the power is small for so large a mill. The shafting of this mill is very heavy, and the grinder is run by a belt from the main shaft. The mill, although on a 43-foot tower, should be at least 20 feet higher. It will be seen to be a very poor mill.

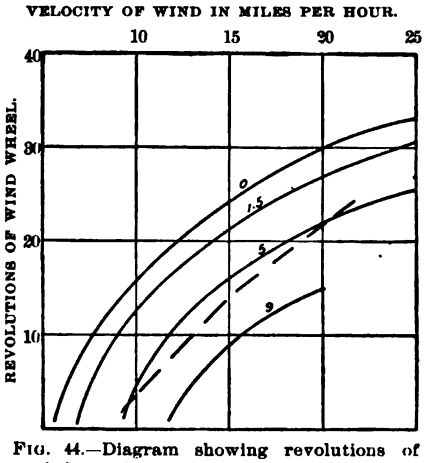

FIG. 44.—Diagram showing revolutions of wind wheel of mill No. 49—22.5-foot wooden Halliday. Curves marked 0, 1.5, 5, and 9 are for brake loads of 0, 1.5, 5, and 9 pounds, respectively; the dotted curve shows the speed of wheel for grinder load.

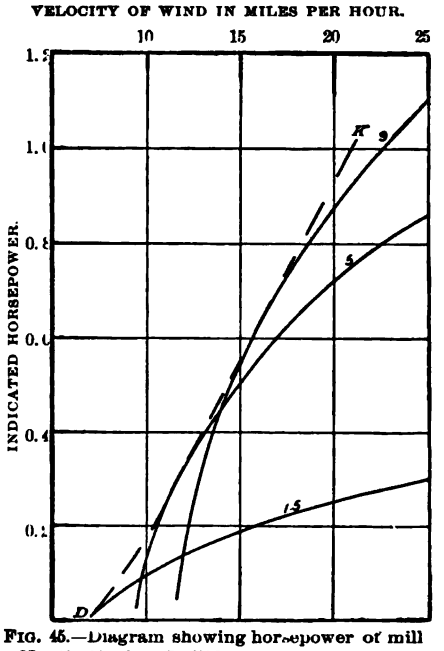

FIG. 45.—Diagram showing horsepower of mill No. 49—22.5-foot Halliday. Curves marked 9, 5, and 1.5 pounds show horsepower for brake loads of 9, 5, and 1.5 pounds, respectively; the dotted line *DK* shows maximum horsepower.

Mill No. 50.—This is a 12-foot Monitor wooden power mill on a 36-foot wooden tower. (See fig. 46.) It is a sectional mill and has 96 sails, each 44 by 4.25 by 1.75 inches, set at an angle of 34° to the plane of the wheel. The shaft is geared forward 3.66 to 1. The swivel gearing, which enables the mill to turn easily and keep full in the wind, is shown in fig. 47. The mill is used for shelling and grinding corn and pumping water. It is in very good condition, and the wind exposure is very good. The mill had been in use about three years. The mean temperature during the time of test was 83° F., the mean barometric pressure 28.6 inches.

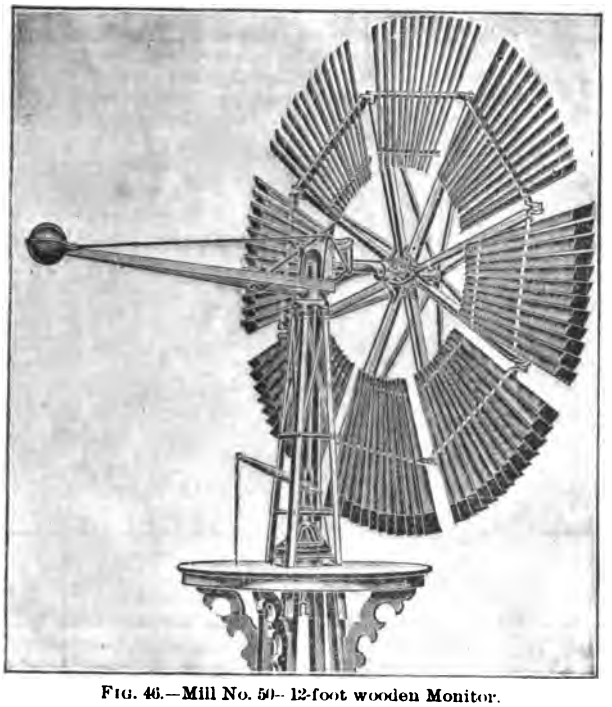

FIG. 46.—Mill No. 50—12-foot wooden Monitor.

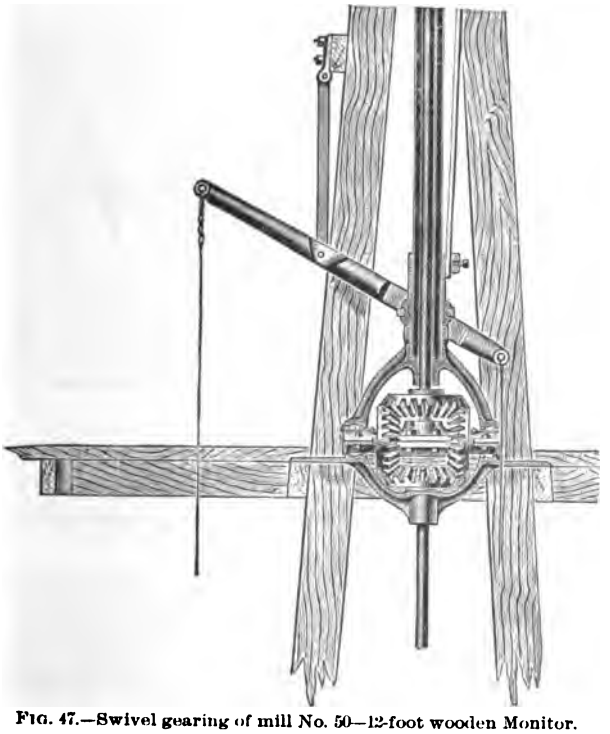

FIG. 47.—Swivel gearing of mill No. 50—12-foot wooden Monitor.

The results of the tests are as follows:

Results of tests of mill No. 50—12-foot wooden Monitor.

Load on brake.	Load per revolution of wind wheel.	Number of revolutions of wind wheel per minute at given wind velocities (per hour).					Horsepower of mill at given wind velocities (per hour).				
		8 miles.	12 miles.	16 miles.	20 miles.	25 miles.	8 miles.	12 miles.	16 miles.	20 miles.	25 miles.
Lbs.	*Ft.-lbs.*										
0	0	16	33	44	54	64	.....	.....	.....	.....	.....
1.5	120	.....	25	35	43	52	.....	0.091	0.127	0.156	0.189
2¾	314	.....	.....	27	34	40	.....	.....	0.230	0.324	0.381
4.5	490	.....	.....	10	24	30	.....	.....	0.150	0.357	0.445

The revolutions of the wind wheel for three brake loads (0, 1.5, and 2.875 pounds) are shown in fig. 48. This figure also shows the num-

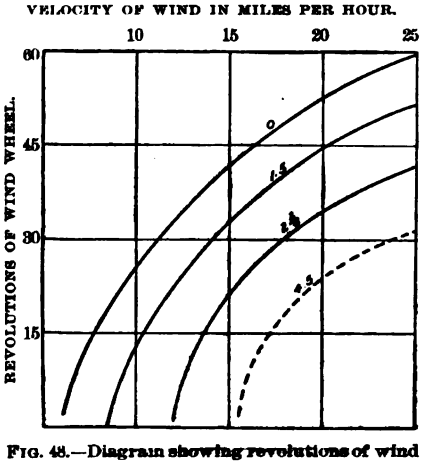

FIG. 48.—Diagram showing revolutions of wind wheel of mill No. 50—12-foot wooden Monitor. Curves marked 0, 1.5, 2¾, 4.5 are for brake loads of 0, 1.5, 2¾, and 4.5 pounds, respectively.

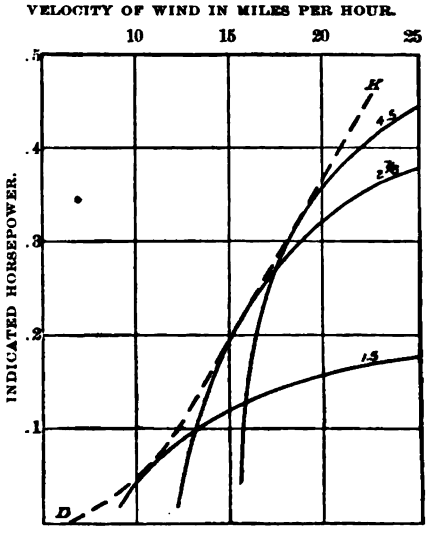

FIG. 49.—Diagram showing horsepower of mill No. 50—12-foot wooden Monitor. Curves marked 4.5, 2¾, and 1.5 are for brake loads of 4.5, 2¾, and 1.5 pounds, respectively; dotted curve *DK* shows maximum power.

ber of revolutions for a 4.5-pound load, found by interpolation from the other results. This mill will be seen to require about a 6-mile wind to start it without any load and to make only 64 revolutions per minute in a 25-mile wind. The 12-foot Aermotor will start in a 4.5-mile wind and make 87 revolutions per minute in a 25-mile wind with no load. Fig. 49 shows the horsepower of this mill for the four loads; also the maximum horsepower.

Mill No. 52.—This is a 14-foot Challenge wooden power mill on a 45-foot wooden tower, manufactured by the Challenge Windmill Company, of Batavia, Illinois. (See fig. 50.) It is a sectional mill, and has two side wheels for keeping the main wheel in the wind. The

wind wheel has 102 sails, each 51.5 by 5 by 1.75 inches, set at an angle of 39° to the plane of the wheel. The mill works a sheller, a grinder, and a pump. There are two horizontal shafts, one of which works the grinder and sheller, the other the pump. The shaft that works the pump is 12 feet long and 1.5 inches in diameter; it is geared forward 1.5 to 1. The shaft that works the grinder is 6 feet long and 1.5 inches in diameter; it is geared forward 15.25 to 1. The well is a drilled well, 192 feet deep. The lift was 180 feet, the discharge 0.25 quart per stroke. The water is pumped into a large box, and passes to watering troughs when needed. The pump has a counterweight which raises on the downstroke and assists in lifting the water on the upstroke. The mean temperature was 92° F., the mean barometric pressure 28.6 inches. The results of the tests are as follows:

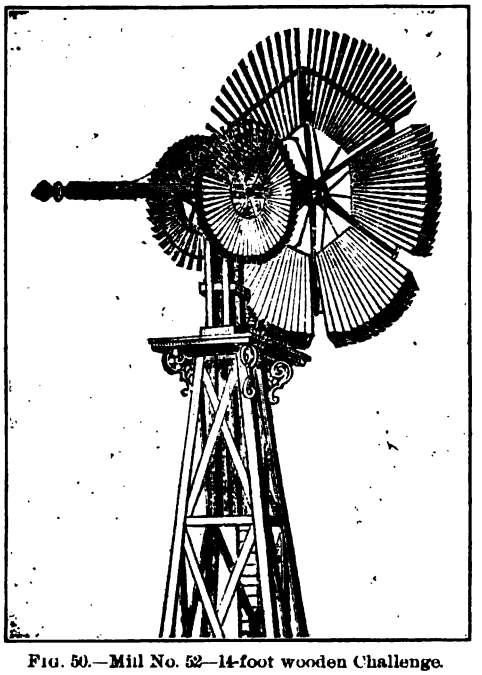

FIG. 50.—Mill No. 52—14-foot wooden Challenge.

Results of tests of mill No. 52—14-foot wooden Challenge.

Load on brake.	Load per revolution of wind wheel.	Number of revolutions of wind wheel per minute at given wind velocities (per hour).					Horsepower of mill at given wind velocities (per hour).				
		8 miles.	12 miles.	16 miles.	20 miles.	25 miles.	8 miles.	12 miles.	16 miles.	20 miles.	25 miles.
Lbs.	*Ft.-lbs.*										
0	0	7	18	25	30	------	------	------	------	------	------
Pump	------	------	14	22	27	------	------	0.059	0.093	0.115	------
1	432	------	------	17	22	------	------	------	0.222	0.287	------
2	864	------	------	8	16	------	------	------	0.210	0.420	------

The revolutions of wind wheel per minute for loads of 0, 1, and 2 pounds are shown in fig. 51. For 0 load the pump shaft was running with the pump detached; the grinder shaft was not working. When pumping the grinder was not working. When the brake loads of 1 and 2 pounds were being used the pump shaft was not working. A curve was obtained giving the speed of the wind wheel for no brake load with the grinder shaft working and with the pump shaft not working. This curve nearly coincided with that for the

pump load, showing that the friction of the grinder shaft was about equal to the pump load. With the pump shaft working, but not the pump, the mill will be seen to require a 7-mile wind to start it, and it makes only 30 revolutions in a 20-mile wind. Fig. 52 shows the horsepower of this mill.

This is a hard-running mill; there is too much friction. The side wheels do not respond to changes in the direction of the wind as quickly

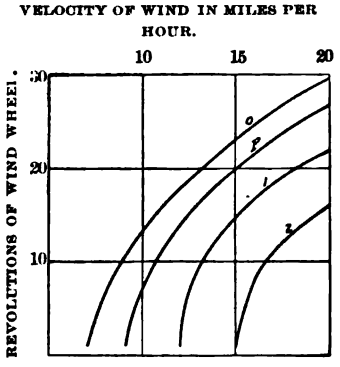

FIG. 51.—Diagram showing revolutions of wind wheel of mill No. 52—14-foot wooden Challenge. Curve marked 0 is for no brake load; curve *P* is for pump load; curves 1 and 2 are for brake loads of 1 and 2 pounds, respectively.

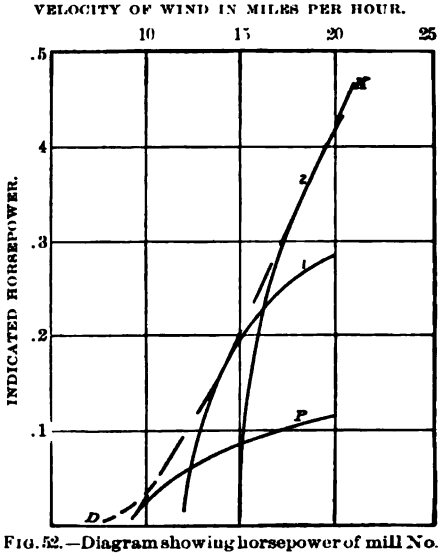

FIG. 52.—Diagram showing horsepower of mill No. 52—14-foot wooden Challenge. Curves marked 1 and 2 are for brake loads of 1 and 2 pounds, respectively; curve *P* is for pump load; dotted curve *DK* shows maximum power.

as does the vane or rudder in other mills. The wind exposure was very good and the mill was nearly new.

Mill No. 53.—This is a 12-foot Ideal power mill on a 33-foot wooden tower. The wind wheel has 21 curved sails, each 43.25 by 16.5 by 8.25 inches, set at an angle of 32° to the plane of the wheel. The horizontal shaft is geared forward 6.07 to 1. The mill had been in use about four years for shelling and grinding corn. Three brake loads were used in the test—0, 1.5, and 2.5 pounds. The mean temperature during the tests was 92° F., the mean barometric pressure 28.7 inches. The results of the tests are as follows:

Results of tests of mill No. 53—12-foot Ideal.

Load on brake.	Load per revolution of wind wheel.	Number of revolutions of wind wheel per minute at given wind velocities (per hour).					Horsepower of mill at given wind velocities (per hour).				
		8 miles.	12 miles.	16 miles.	20 miles.	25 miles.	8 miles.	12 miles.	16 miles.	20 miles.	25 miles.
Pounds.	*Ft.-lbs.*										
0	0	25	40	51	60	68	.....	.....	.....	.....	.....
1.5	272	.....	29	41	51	60	.....	0.239	0.338	0.420	0.500
2.5	455	.....	.....	32	44	54	.....	.....	0.440	0.606	0.745

Fig. 53 shows the number of revolutions per minute of the wind wheel for the three brake loads. The mill will be seen to start in about a 5-mile wind and to make 69 revolutions a minute in a 25-mile wind with no load. Fig. 54 shows the horsepower for the several loads.

Mill No. 54.—This is a 12-foot Aermotor like No. 27, on a 47-foot tower. The 12-foot Aermotor No. 27 was found to be so much greater in power and speed than other 12-foot mills tested that we thought

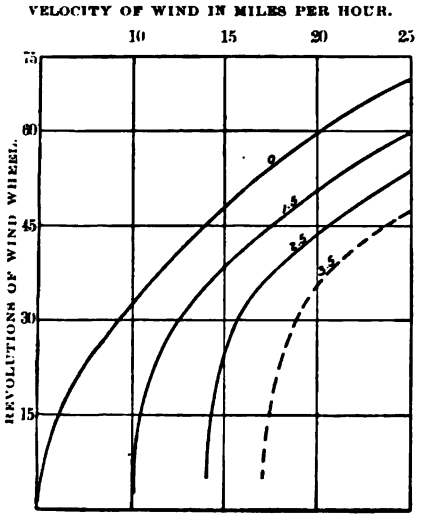

FIG. 53.—Diagram showing revolutions of wind wheel of mill No. 53—12-foot Ideal. Curves marked 0, 1.5, 2.5, and 3.5 are for brake loads of 0, 1.5, 2.5, and 3.5 pounds, respectively.

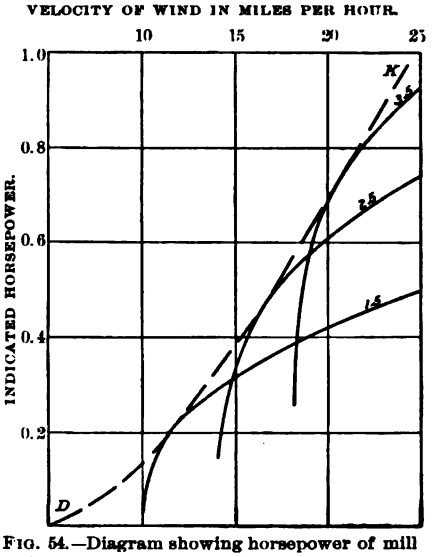

FIG. 54.—Diagram showing horsepower of mill No. 53—12-foot Ideal. Curves marked 3.5, 2.5, and 1.5 are for brake loads of 3.5, 2.5, and 1.5 pounds, respectively; dotted curve *DK* shows maximum power.

it wise to test another of the same make and size under somewhat different conditions. No. 54 had been in use about two years. Two brake loads were used—0 and 297 foot-pounds per revolution of wind wheel. The mean temperature during the test was 92° F., the mean barometric pressure 29.2 inches. The results of the tests are as follows:

Results of tests of mill No. 54—12-foot Aermotor.

Load on brake.	Load per revolution of wind wheel.	Number of revolutions of wind wheel per minute at given wind velocities (per hour).					Horsepower at given wind velocities (per hour).				
		8 miles.	12 miles.	16 miles.	20 miles.	25 miles.	8 miles.	12 miles.	16 miles.	20 miles.	25 miles.
Pounds.	*Ft.-lbs.*										
0	0	31	49	63	75	87	.....	.....	.....	.....	.....
1.75	297	.....	38	52	63	75	.....	0.34	0.46	0.57	0.68

It will be seen that the results of the tests of this mill apparently agree closely with those of No. 27, page 86. The agreement, however,

is not so close as it appears, since the temperature for No. 54 is much higher. It is, however, about the same as the temperature for other mills, so that we can still use the results found for mill No. 27 in comparing its power and speed with those of other mills.

COMPARISON OF POWER MILLS.

Comparison of 12-foot Ideal (No. 53) with 14-foot Ideal (No. 34).—It must be remembered that in this and in all other comparisons no correction is made for difference in temperature and barometric pressure. The speeds for no load and the maximum horsepowers for these mills are as follows:

Comparison of results for 12-foot Ideal and 14-foot Ideal.

Mill.	Number of revolutions of wind wheel per minute at given wind velocities (per hour).					Maximum horsepower at given wind velocities (per hour).				
	8 miles.	12 miles.	16 miles.	20 miles.	25 miles.	8 miles.	12 miles.	16 miles.	20 miles.	25 miles.
12-foot Ideal	25	40	51	60	68	0.06	0.23	0.44	0.70	.....
14-foot Ideal	20	34	44	53	58	0.06	0.22	0.46	0.74	.....
Ratio of circumference velocities ($\frac{12}{14}$)	1.07	1.00	0.99	0.97	1.00	.....	.....	.....	.....	.....

It will be seen that the useful power of the 14-foot mill is very little more than that of the 12-foot mill, and that the circumference velocities are nearly the same for no load, where no horizontal shaft is being turned. In the case of the 12-foot mill the brake was on the foot gear and there was no horizontal shaft to turn, but in the case of the 14-foot mill the brake pulley was on a shaft 15 or 20 feet long. We believe that if the brake pulley had been on the foot gear and the line shaft thrown out of gear, so as to eliminate shaft friction, the mill would have shown at least 10 per cent more power. The tower obstructs the wheel somewhat and reduces the power.

Comparison of 12-foot Aermotor (No. 27) with 14-foot Ideal (No. 34).—The speeds for no load and the maximum horsepowers for these mills are as follows:

Comparison of results for 12-foot Aermotor and 14-foot Ideal.

Mill.	Number of revolutions of wind wheel per minute at given wind velocities (per hour).					Maximum horsepower at given wind velocities (per hour).			
	8 miles.	12 miles.	16 miles.	20 miles.	25 miles.	8 miles.	12 miles.	16 miles.	20 miles.
12-foot Aermotor	30	49	63	75	87	0.09	0.33	0.66	1.05
14-foot Ideal	20	34	44	53	58	0.06	0.22	0.46	0.74
Ratio of circumference velocities ($\frac{12}{14}$)	1.29	1.23	1.22	1.21	1.28	.....	.....	.....	.....

It will be seen that the number of revolutions per minute of the 12-foot mill is 50 per cent greater than for the 14-foot mill. This is true for the lower as well as for the higher velocities, where the governing of the mill does not enter to reduce the speed. It will be seen also that the 12-foot mill is producing from 42 to 50 per cent more horsepower than the 14-foot mill. The temperature was 49° higher and the pressure 0.6 inch lower when the 14-foot mill was tested than when the 12-foot mill was tested. The effect is to lessen the difference between the power and speeds of the mills.

Comparison of 12-foot Aermotor (No. 27) with 16-foot Aermotor (No. 44).—Attention has already been drawn to the similarity between the speed curves of these mills (figs. 35 and 42) and between the power curves. (The power curves for mill No. 44 are shown in fig. 43; those for mill No. 27 were platted, but are not published because of lack of space.) If we compare the number of revolutions per minute for no load, we shall see that they are to each other nearly inversely as the diameter, or that the circumference velocities of the two mills are the same in all wind velocities. We may compare the brake horsepower and the speed as follows:

Comparison of results for 12-foot Aermotor and 16-foot Aermotor.

Mill.	Number of revolutions of wind wheel per minute at given wind velocities (per hour).					Maximum horsepower at given wind velocities (per hour).		
	8 miles.	12 miles.	16 miles.	20 miles.	25 miles.	10 miles.	15 miles.	20 miles.
12-foot Aermotor	30	49	63	75	87.0	0.21	0.58	1.05
16-foot Aermotor	23	38	48	56	64.5	0.29	0.82	1.55
Ratio of circumference velocities ($\frac{12}{16}$)	1.02	1.03	1.02	0.99	0.99	1.38	1.41	1.48

From this it will be seen that the power of the 16-foot Aermotor is about 1.22 times that of the 12-foot. The ratio of the squares of the diameters is 1.78; the ratio of the diameters 1.33. It will be seen that the power does not increase as the squares of the diameters, as is often stated; it increases faster than as the diameters, but more nearly as the diameters than as the squares of the diameters.

Comparison of 16-foot Althouse wooden mill (No. 28) with 16-foot Aermotor (No. 44).—From the following table it will be seen that the wind wheel of No. 44 is revolving from 56 to 77 per cent faster than the wind wheel of No. 28. The latter, however, has to overcome the friction of 10 or 12 feet of line shafting. It will be seen that No. 44 is yielding from 70 to 167 per cent more power than No. 28. The superiority of the steel mill over the wooden mill is very evident in this case.

Comparison of results for 16-foot Althouse and 16-foot Aermotor.

Mill.	Number of revolutions of wind wheel per minute at given wind velocities (per hour).					Maximum horsepower at given wind velocities (per hour).			
	8 miles.	12 miles.	16 miles.	20 miles.	25 miles.	8 miles.	12 miles.	16 miles.	20 miles.
16-foot Althouse	13	23	30	36	40.0	0.06	0.29	0.52	.....
16-foot Aermotor	23	38	48	56	64.5	0.16	0.48	0.93	1.55
Ratio of circumference velocities $\left(\frac{\text{steel}}{\text{wood}}\right)$	1.77	1.65	1.60	1.55	1.61	2.67	1.65	1.78	.....

Comparison of 22.5-foot Halliday wooden mill (No. 49) with 16-foot Aermotor (No. 44).—From the following table it will be seen that the steel mill makes about two revolutions to one revolution of the wooden mill, and that its power is from 41 to 167 per cent greater.

Comparison of results for 22.5-foot Halliday and 16-foot Aermotor.

Mill.	Number of revolutions of wind wheel per minute at given wind velocities (per hour).				Maximum horsepower at given wind velocities (per hour).			
	8 miles.	12 miles.	16 miles.	20 miles.	8 miles.	12 miles.	16 miles.	20 miles.
22.5-foot Halliday	11	20	25	30	0.06	0.34	0.62	0.88
16-foot Aermotor	23	38	48	56	0.16	0.48	0.93	1.55
Ratio of circumference velocities $\left(\frac{16}{22.5}\right)$	1.48	1.35	1.36	1.32	2.67	1.41	1.50	1.76

Comparison of 12-foot Aermotor (No. 27) with 22.5-foot Halliday wooden mill (No. 49).—Comparing the power of these mills, we have the following:

Comparison of results for 12-foot Aermotor and 22.5-foot Halliday.

Mill.	Number of revolutions of wind wheel per minute at given wind velocities (per hour).				Maximum horsepower at given wind velocities (per hour).			
	8 miles.	12 miles.	16 miles.	20 miles.	8 miles.	12 miles.	16 miles.	20 miles.
12-foot Aermotor	30	49	63	75	0.09	0.33	0.66	1.05
22.5-foot Halliday	11	20	25	30	0.06	0.32	0.63	0.94
Ratio of circumference velocities $\left(\frac{12}{22.5}\right)$	1.46	1.31	1.34	1.32	1.50	1.03	1.05	1.12

It will be seen that this 22.5-foot wooden mill does not furnish as much power as a good 12-foot steel mill.

Comparison of wooden power mills.—Of the mills in the following table the 12-foot has the least friction; the tower being in front of the windmill obstructs the wheel and reduces the power. The 14-foot mill probably has more friction than the others. The 16-foot Irrigator has too few sails; with more sails the power could probably be increased 75 per cent or more.

Comparison of results of tests of wooden power mills.

Mill.	Number of revolutions of wind wheel per minute at given wind velocities (per hour).					Horsepower at given wind velocities (per hour).			
	8 miles.	12 miles.	16 miles.	20 miles.	25 miles.	8 miles.	12 miles.	16 miles.	20 miles.
12-foot Monitor (No. 50)	16	33	44	54	64	0.02	0.10	0.23	0.38
14-foot Challenge (No. 53)	7	18	25	30	.....	0.01	0.10	0.25	0.42
16-foot Althouse (No. 23)	13	23	30	36	40	0.06	0.29	0.52	0.84
16-foot Irrigator (No. 30)	12	25	32	41	44	0.02	0.16	0.30	0.44
22.5-foot Halliday (No. 49)	11	20	25	30	.....	0.06	0.32	0.63	0.94

It will be seen that the 16-foot mill (No. 28) is furnishing from 2.2 to 2.9 times more useful power than the 12-foot mill (No. 50). It will also be seen that the power of this 16-foot mill compared with that of the 12-foot mill increases faster than as the squares of the diameters, while the power of the 22.5-foot mill compared with that of the 12-foot mill does not increase as fast as the squares of the diameters, and the power of the 22.5-foot mill compared with that of the 16-foot mill does not increase as fast as the first power of the diameters.

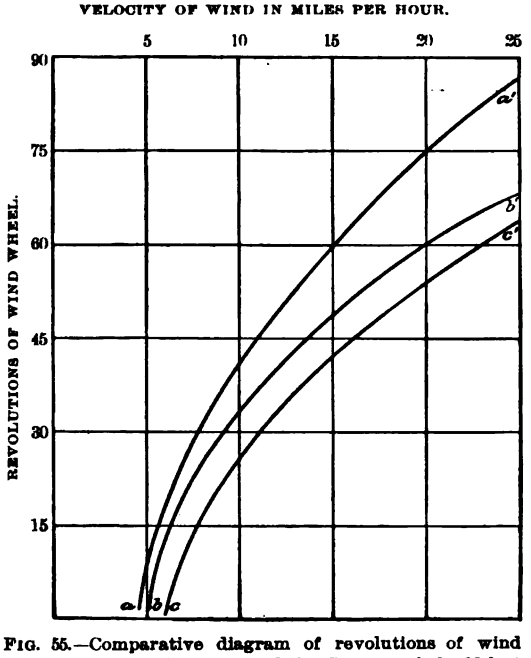

FIG. 55.—Comparative diagram of revolutions of wind wheels of mills Nos. 50, 53, and 27. Curve *aa'* is for 12-foot Aermotor; *bb'* is for 12-foot Ideal; *cc'* is for 12-foot Monitor.

Comparison of 12-foot Monitor wooden mill (No. 50) with 12-foot Aermotor (No. 27) and with 12-foot Ideal (No. 53).—Fig. 55 shows the speed, in revolutions, of the wind wheels of these mills for no useful load. The friction is small in each case. The Monitor has a swivel gearing, and the Ideal has a ball

gearing to carry the weight of the shaft, both of which reduce the friction. It is fair to say that the friction load of the Aermotor is at least as great as that of the other mills. It will be seen that the speed of the Ideal is noticeably greater than that of the Monitor; and that the speed of the Aermotor is much greater than that of the Ideal, especially for high wind velocities. This is the reason the Aermotor is so much more powerful than other mills. It revolves much faster, and the power is directly proportional to the speed. But why is its speed greater for the same load?

Comparison of results for 12-foot Monitor, 12-foot Ideal, and 12-foot Aermotor.

Mill.	Load per revolution of wind wheel.	Number of revolutions of wind wheel per minute at given wind velocities (per hour).					Horsepower at given wind velocities (per hour).				
		8 miles.	12 miles.	16 miles.	20 miles.	25 miles.	8 miles.	12 miles.	16 miles.	20 miles.	25 miles.
	Ft.-lbs.										
12-foot Monitor	0	16	33	44	54	64	.....	.....	.....	.....	.....
	120	.....	25	35	43	52	.....	0.091	0.127	0.156	0.189
	314	.....	.....	27	34	40	.....	.....	0.230	0.324	0.381
12-foot Ideal	0	25	40	51	60	68	.....	.....	.....	.....	.....
	272	.....	49	41	51	60	.....	0.239	0.338	0.420	0.500
	455	.....	.....	32	44	54	.....	.....	0.440	0.606	0.745
12-foot Aermotor	0	30	29	63	75	87	.....	.....	.....	.....	.....
	222	16	43	57	70	81	0.089	0.285	0.386	0.458	0.523
	444	.....	23	48	65	77	.....	0.303	0.653	0.890	1.020
	666	.....	.....	12	50	72	.....	.....	0.234	1.028	1.451

The dimensions of the principal parts of the wind wheels of these mills and the mean temperature and pressure when the tests were made are as follows:

Dimensions of principal parts of mills Nos. 50, 53, and 27.

Mill.	Number of sails.	Dimensions of sails.	Angle of sails.	Gearing.	Mean temperature.	Mean barometric pressure.
		Inches.	°		° *F.*	*Inches.*
12-foot Monitor (No. 50)	96	44 x 4¼x1¼	34	3.66 : 1	83	28.6
12-foot Ideal (No. 53)	21	43¼x16¼x8¼	32	6.07 : 1	92	28.7
12-foot Aermotor (No. 27)	18	44 x18¼x7¼	31	6.00 : 1	46	28.9

The temperature when the 12-foot Aermotor was tested was much lower than when the other two mills were tested, but, as already stated, the temperature when Aermotor No. 54 was tested was 92° F., and it showed a speed and power about the same as Aermotor No. 27; so we may leave this difference in temperature out of account.

It will be seen that the Monitor is not geared forward as much as the other two, but this does not affect the speed for no load. The sail angle is about the same for all, also the length of sail; but the

number of sails and the width are quite different. The Aermotor has few sails, but of large size; the Ideal has more and somewhat smaller sails; the Monitor has many small sails, and its tower is located in front of the wind wheel. Its sails are plane, instead of curved, all of which tends to decrease its power.

Fig. 56 shows the curves of maximum horsepower. It will be seen that the difference between the horsepower of the Aermotor and that of the Ideal is about the same as the difference in their speeds for the same wind velocity, but the difference between the power of the Monitor and that of the Ideal is much greater than the corresponding difference in their speeds. The wooden mill, therefore, not only has a less speed at a given wind velocity than the steel mill, but it carries a proportionately less load. For example, in a 20-mile wind a load of 120 foot-pounds per revolution reduces the speed of the Monitor from 54 to 43 (or 11) revolutions per minute, while in the case of the Aermotor a load of 220 foot-pounds (about 80 per cent greater) reduces the speed from 75 to 70 (only 5) revolutions per minute. In the case of the Ideal a load of 272 foot-pounds per revolution (2.3 times the load of the Monitor) reduces the speed from 60 to 51 (or 9) revolutions per minute.

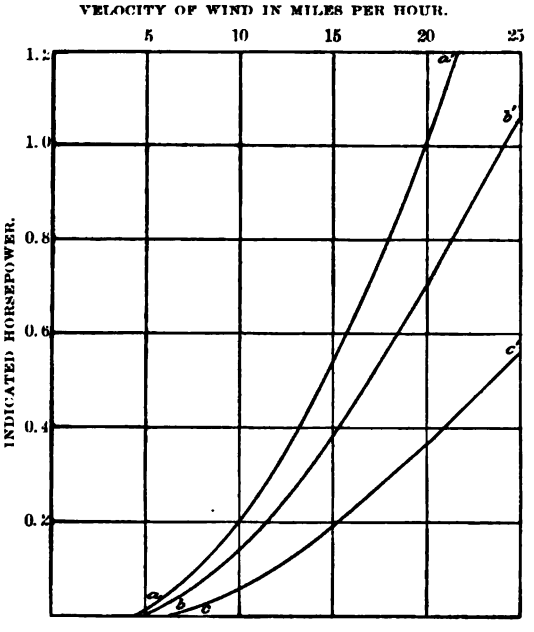

FIG. 56.—Comparative diagram of horsepower of mills Nos. 50, 53, and 27. Curve *aa'* is for 12-foot Aermotor; *bb'* is for 12-foot Ideal; *cc'* is for 12-foot wooden Monitor.

COMPARISON OF PUMPING MILLS WITH POWER MILLS.

Comparison of 12-foot pumping mill (No. 3) with 12-foot power mill (No. 27).—The load per stroke of No. 3 (see page 30, Part I) is 415.3 foot-pounds. The wind wheel makes 3.3 revolutions to 1 stroke of the pump, so that the load per revolution of wind wheel is 124.5 foot-pounds. This is less than the smallest load used in testing No. 27, viz, 222 foot-pounds per revolution. A diagram was platted showing the useful work, in horsepower, of these mills for these loads. The curve for the pumping mill was seen to start with a little less wind velocity than that of the power mill, indicating a somewhat less total load.

Comparing the ordinates of these curves for different wind velocities, the following ratios were obtained, which give, approximately, the pump efficiency, no allowance being made for difference in temperature and pressure:

8	12	16	20	25	30
0.60	0.53	0.54	0.54	0.58	0.58

The mean of these ratios is 0.56. If the useful load of the pumping mill had been somewhat greater, so that the mills would have started at the same wind velocity, the ratio, or pump efficiency, would be about 60 per cent, which is about what might be expected of this pump under this lift. The ratio of the useful loads is $125 \div 222 = 0.57$. This ratio would probably be about 0.60 if the loads were such that the mills would start at the same wind velocity.

Comparison of 16-foot pumping Aermotor (No. 9) with 16-foot power Aermotor (No. 44).—The useful load of No. 9 (see pages 36 to 37, Part I) is 1,013 foot-pounds per stroke of pump, or 304 foot-pounds per revolution of wind wheel. The smallest useful load of No. 44 is 528 foot-pounds per revolution of wind wheel. A diagram was platted showing the useful horsepower of these mills for these loads. For this particular load (1,013 foot-pounds) the pumping mill was seen to start at a somewhat less wind velocity than the power mill, indicating that the total load of the pumping mill was somewhat less than that of the power mill. The ratio of any two of the ordinates gave, approximately, the pump efficiency for that wind velocity, the difference in temperature and pressure being neglected.

These ratios for four velocities are as follows:

12	16	20	25
0.72	0.67	0.69	0.63

If the pump load had been somewhat greater—such that the mills would start at the same wind velocity—the mean ratio would be about 70 per cent. This, again, is about what is expected for the efficiency of this pump, which is somewhat better than mill No. 3 and has a higher lift. The ratio of the useful loads of these mills is $304 \div 528 = 0.57$. This ratio would probably be about 60 per cent if the mills started at the same wind velocity.

It is interesting to compare the performance of these mills still further. The speeds of the wheels and the horsepowers are as follows:

Comparison of results for 16-foot pumping Aermotor and 16-foot power Aermotor.

Mill.	Number of revolutions of wind wheel per minute at given wind velocities (per hour).				Horsepower at given wind velocities (per hour).				Load per revolution of wind wheel.
	12 miles.	16 miles.	20 miles.	25 miles.	12 miles.	16 miles.	20 miles.	25 miles.	
									Ft.-lbs.
16-foot pumping mill (No. 9)	31	42	52	59	0.325	0.433	0.548	0.601	0.304
16-foot power mill (No. 44)	28	41	50	58	0.45	0.65	0.80	0.95	0.528
Ratio of pump power to mill power	------	------	------	------	0.72	0.67	0.69	0.63	0.58

It will be seen that the wind wheel of the pumping mill is making from one to two more revolutions per minute than that of the power mill.

Comparison of 22.5-foot pumping mill (No. 36) with 22.5-foot power mill (No. 49).—In this comparison we will use the curve of 5 pounds, or 1,087 foot-pounds, per revolution of wind wheel as the speed for this load, corresponding more nearly with that for the pump load than any other. The speeds of the mills and the horsepowers are as follows:

Comparison of results for 22.5-foot pumping mill and 22.5-foot power mill.

Mill.	Number of revolutions of wind wheel per minute at given wind velocities (per hour).				Horsepower at given wind velocities (per hour).				Load per revolution of wind wheel.
	12 miles.	16 miles.	20 miles.	25 miles.	12 miles.	16 miles.	20 miles.	25 miles.	
									Ft.-lbs.
22.5-foot pumping mill (No. 36)	12	17	20	24	0.090	0.124	0.150	0.182	0.248
22.5-foot power mill (No. 49)	11	18	22	26	0.342	0.593	0.724	0.856	1.087
Ratio of pump power to mill power	------	------	------	------	0.26	0.20	0.20	0.22	0.23

The efficiency of the pump is, therefore, not more than 22 per cent. The ratio of useful loads is 22 per cent. (For description of pumping mill No. 36, see pages 52 to 53, Part I.)

EFFECT OF TENSION OF SPRING ON SPEED AND HORSEPOWER OF MILL.

The effect of tightening the spring which holds the wind wheel of mill No. 18 in the wind has been shown on page 44, Part I. The effect of a reduction in the tension of the spring of the 16-foot power mill (No. 44) is shown in fig. 57. The curve *aa'* is for no load and the spring new and stiff. The curve *aa''* is for the same load (none) and spring after the mill had been out of use about eight months. It will be seen that these curves coincide up to a velocity of about 10 miles an hour, after which they separate rapidly. In a 25-mile wind the number of revolutions of the wind wheel per minute has been reduced from 64.5 to 54 by the decrease in the tension of the spring. The curve *bb'* is for a 3-pound brake load with relaxed spring. It will be seen to be nearly parallel to the curve *aa''*, showing that the effect of the load when the spring is relaxed is similar to that when it is taut. We see how important a factor tension of spring is on the power of a mill. It also shows that if a spring is to be used in place of a weight there should be some easy way to change its tension.

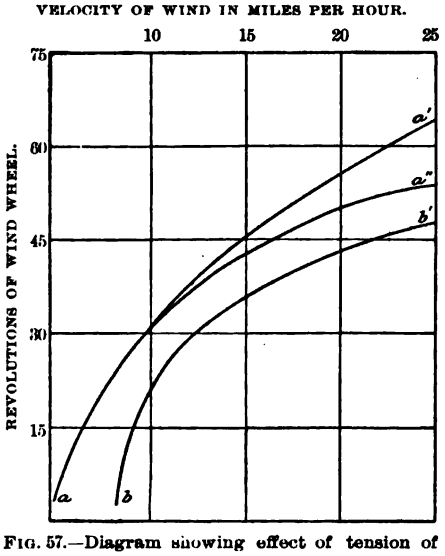

FIG. 57.—Diagram showing effect of tension of spring of mill No. 44—16-foot Aermotor. Curve *aa'* is for no load, spring new and stiff; *aa''* is for no load, spring relaxed, mill having been out of use eight months; *bb'* is for 3-pound brake load with relaxed spring.

MATHEMATICAL DISCUSSION OF TESTS OF TWO AERMOTORS.

In this discussion the wind velocities are those found by the use of the Robinson cup anemometer. A comparison of these velocities with true wind velocities will be given later. The Aermotors are selected for this discussion because their power is greater than that of any other form of mill that we have tested, and because their speed and power curves are derived from a greater number of observations than those of any other mill.

Discussion of tests of 12-foot Aermotor (No. 27).—The curves showing the number of revolutions of wind wheel (called speed curves) of this mill for four brake loads are given on page 88—fig. 35. Each of these curves is seen to resemble a parabola the axis of which is the x coordinate axis on which the wind velocities are marked. Each of these has the form $y^2 = a + bx$, in which x is the wind velocity in miles per

hour, y the speed of wind wheel in revolutions per minute, and a and b constants. For the curve of no load (0) we have $y = 50$ when $x = 12$, and $y = 75$ when $x = 20$. Hence we have

$$50^2 = a + 12b, \text{ and}$$
$$75^2 = a + 20b.$$

Solving these, for a and b we have $a = -2{,}187$ and $b = 391$; and the equation of the curve is

$$y^2 = -2{,}187 + 391x. \tag{1}$$

For the speed curve of 2 pounds we see that $y = 43$ when $x = 12$, and that $y = 70$ when $x = 20$. Hence we have

$$43^2 = a + 12b, \text{ and}$$
$$70^2 = a + 20b.$$

Solving these, we have $a = -2{,}728$ and $b = 381$; and we have the equation of this 2-pound curve

$$y^2 = -2{,}728 + 381x. \tag{2}$$

Proceeding in a similar way, we have for the equation of the 4-pound curve

$$y^2 = -4{,}384 + 424x. \tag{3}$$

For the equation of the 6-pound curve we have

$$y^2 = -9{,}400 + 595x. \tag{4}$$

The speed as determined by measurement and as found from these equations for several wind velocities is shown the following table.

The starting velocities are found by making $y = 0$ and solving for x in equations 1 to 4.

Table showing revolutions per minute of 12-foot Aermotor (No. 27) under different loads and at different wind velocities.

Wind velocity per hour.	No load.		2-pound load.		4-pound load.		6-pound load.	
	Measured.	Computed.	Measured.	Computed.	Measured.	Computed.	Measured.	Computed.
	Rev.	*Rev.*	*Rev.*	*Rev.*	*Rev.*	*Rev.*	*Rev.*	*Rev.*
8 miles	30	30	16	18	.....	.....	.....	.....
12 miles	49	50	43	43	23	27	.....	.....
16 miles	63	64	57	58	48	49	12	10
20 miles	75	75	70	70	65	64	50	50
25 miles	87	87	81	82	77	78	72	74
Starting velocity	4.5	5.6	7.0	7.1	10.2	10.3	15.3	15.8

If the origin of coordinates for equation 1 be moved to the point where the curve crosses the axis of x, the equation will then be of the

form $y^2 = 391x'$, from which we have $y = \sqrt{391x'}$; that is, the speed for a constant load increases as the square root of the wind velocity.

The close agreement between the measured and computed speeds, especially for the curve of no load, is noticeable. The measured and computed starting velocities differ somewhat. This was expected, since it is difficult to get the starting velocities from observation.

Hereafter in this discussion the computed instead of the observed speeds and starting velocities will be used. It must be remembered, however, that these are not what may be called theoretical results. They are obtained from measurements, not from theory, and are the adjusted values of the observed quantities.

The power curves for this mill for three brake loads were platted, but are not published because of lack of space. The curves are parabolas with their axes horizontal. This follows at once from the fact that the corresponding speed curves are parabolas. The power is proportional to the product of the load and speed. When the load is constant, as it is for one of these speed curves, the power varies as the speed, and hence the load curves are parabolas.

The equation of any one of the curves—as, for example, the 2-pound curve—may be found as follows: The formula for horsepower is $\text{H. P.} = 2\pi RuL \div 33{,}000$, $R = 35.5 \div 12$ and $L = 2$. Hence $\text{H. P.} = 2 \times \overline{22 \div 7} \times 6 \times 35.5 \times 2 \times u \div \overline{12 \times 33{,}000} = 0.0067u = Ku$ where $K = 0.0067$.

In the speed equations u is what we have called y, and $y = -2{,}728 + 381x$. Hence—

$$\text{H. P.} = Ky = K\sqrt{-2{,}728 + 381x}, \text{ and}$$
$$(\text{H. P.})^2 = K^2(-2{,}728 + 381x) = -0.1225 + 0.017x. \qquad (5)$$

In the diagram of power curves, platted but not reproduced here, the curve of maximum power was found to resemble a parabola the axis of which was vertical with its vertex on the y coordinate axis below the origin. The form of its equation is $x^2 = a + by$, x being the wind velocity, in miles per hour, y the horsepower, and a and b constants. For $x = 5$, $y = 0$, and for $x = 20$, $y = 1.05$. Substituting these values in the above equation we have: $25 = a$, and $400 = a + 1.05b$. From these we have: $a = 25$, $b = 357$, and the equation of the maximum power curve is—

$$x^2 = 25 + 357y. \qquad (6)$$

For $x = 5$, 10, 15, 20, and 25, y has the values 0, 0.21, 0.56, 1.05, and 1.69, which agree closely with those taken from the curve.

For the mill to yield the greatest amount of power possible the load should increase as the wind velocity increases. In an 8.5-mile wind a 2-pound load gives the maximum power; in a 14-mile wind a 4-pound load gives the maximum power, and in a 21-mile wind a 6-pound load gives the maximum power.

We wish to determine how the load and speed of the mill vary with

the wind velocity for the curve of maximum power. The load curves were found to be tangent to the curve of maximum power for loads and velocities about as follows: The 0 curve is tangent at $x = 5$, the 2-pound curve at $x = 8.5$, the 4-pound curve at $x = 21$. For a constant increment of 2 pounds in the load the increment of wind velocity changed from 3.5 miles to 7 miles. Hence the velocity increases faster than the loading. For each of these four points on the curve the load and horsepower are known. Hence we can find the number of revolutions from the equation—

$$\text{H. P.} = 2 \times \pi \times R \times 6 \times 4 \times L \div 33{,}000. \qquad (7)$$

The wind velocities, loads, powers, and speeds for these four points of tangency are as follows:

Data regarding points of tangency of power curves with curves of maximum power of Aermotor No. 27.

Wind velocity per hour.	Load.	Horsepower.	Revolutions per minute.
	Pounds.		
5 miles	0	0	0
8.5 miles	2	0.13	19
14 miles	4	0.50	38
21 miles	6	1.15	57

The speeds, in revolutions, as here given are platted in fig. 35, giving the curve PQ, which is the speed curve for maximum power. The proper load for maximum power can now be found for any wind velocity from equation 7, the speed being taken from this speed curve. The ratio of the speed at maximum load to the speed at 0 load, for the wind velocities 10, 15, and 20 miles an hour, is 0.63, 0.70, and 0.74, respectively, showing quite an increase. The following table gives additional information in regard to speed and load for the curve of maximum power:

Data regarding speed and load for curve of maximum power of Aermotor No. 27.

Wind velocity per hour.	Load per revolution of wind wheel.	Revolutions per minute.	L − L′.	S − S′.	LS.	LS − L′S′.	Δ^2.
	Pounds.						
5 miles	0	0	.....	.....	.....	.....	.....
10 miles	2.1	25	2.1	25	52.5	52.5	.....
15 miles	4.0	41	1.9	16	160.4	107.9	55.4
20 miles	5.9	55	1.9	14	324.5	164.1	56.2

The fourth column gives the differences between the successive loads, or the increments of loading. These are seen to decrease somewhat, showing that the load does not increase quite as fast as the wind velocity. The fifth column gives the differences between the successive speeds, and shows that the speed does not increase quite

as fast as the wind velocity. The sixth column gives the products of the loads and speeds, which is proportional to the horsepower. The seventh column gives the increments of horsepower, the eighth column the difference between the figures in the seventh column. These, being nearly constant, show that the curve of maximum power is of the second degree.

The following table contains additional interesting information in regard to the speed of this mill:

Data in regard to speed of mill No. 27—12-foot Aermotor.

Wind velocity per hour.	Revolutions per minute, no load.	Circumference velocity, in miles, no load.	Ratio of circumference velocity to wind velocity, no load.	Revolutions per minute at maximum load.	Ratio of speed at maximum load to speed at no load.
8 miles	30	12.9	1.61	17	0.57
12 miles	49	21.0	1.75	32	0.65
16 miles	63	27.0	1.70	44	0.70
20 miles	75	32.1	1.60	54	0.72
25 miles	87	37.3	1.50	.....	.....

The results obtained from this 12-foot mill may be stated as follows, in terms of cup anemometer velocities:

(1) The speed of the wheel for a constant load varies as the square root of the wind velocity.

(2) The speed of the wheel for maximum load increases slightly faster than the first power of the wind velocity.

(3) The power of the mill for a constant load varies as the square root of the wind velocity.

(4) The maximum power of the mill varies as the square of the wind velocity.

(5) The load for maximum power does not increase quite as fast as the wind velocity.

(6) The ratio of speed for maximum load to the speed for no load increases somewhat with the wind velocity.

Discussion of tests of 16-foot Aermotor No. 44.—The speed curves for this mill are shown in fig. 42. They are seen to resemble the parabolas with horizontal axis. The equation of each has the form $y^2=a+bx$, y being the speed in revolutions per minute, x the wind velocity in miles per hour, and a and b being constants for any curve. We see that for $x=12$, $y=38$, and that for $x=20$, $y=56$. Hence we have

$$38^2=a+12b, \text{ and}$$
$$56^2=a+20b.$$

Solving these equations, we have $a=-1{,}094$, $b=211.5$, and the equation of the no-load speed curve is

$$y=-1{,}094+211.5x. \qquad \text{(8)}$$

Proceeding in a similar way, we have for the equation of the 3-pound load speed curve

$$y^2=-1{,}790+214.5x. \tag{9}$$

For the 5-pound load we have

$$y^2=-2{,}304+212x. \tag{10}$$

For the 8-pound load we have

$$y^2=-2{,}715+197x. \tag{11}$$

The speed and starting velocities as computed from these equations and as found by measurement are as follows:

Speed and starting velocities for 16-foot Aermotor No. 44.

Wind velocity per hour.	No load.		3-pound load.		5-pound load.		8-pound load.	
	Measured.	Computed.	Measured.	Computed.	Measured.	Computed.	Measured.	Computed.
8 miles	23.0	24.0	.....	.....	.....	.....	.....	.....
12 miles	38.0	38.0	28.0	28.0	13	15.0	.....	.....
16 miles	48.0	48.0	41.0	40.0	33	33.0	16.0	20.0
20 miles	56.0	56.0	50.0	50.0	44	44.0	36.0	35.0
25 miles	64.5	65.0	59.0	60.0	54	54.0	47.0	47.0
Starting velocity	4.5	5.1	8.0	8.3	11	10.9	14.5	13.8

The computed values are seen to agree closely with the measured values, so that these speed curves are parabolas of the form $y=\sqrt{a+bx}$. The power curves shown in fig. 43 are parabolas of this form for the reason given for the corresponding case of the 12-foot Aermotor. The curve of maximum power to which these power curves are tangent is a parabola with its axis vertical. Its equation has the form $x^2=a+by$. We may obtain the data for finding the value of a and b by observing that when $x=10$, $y=0.30$; and that when $x=20$, $y=1.55$.
We have

$$10^2=a+0.30b, \text{ and}$$
$$20^2=a+1.55b.$$

Solving these, we have $a=28$, $b=240$, and the equation of the curve is

$$x^2=28+240y. \tag{12}$$

The values of y for four values of x are $x=8$, $y=15$; $x=12$, $y=0.48$; $x=16$, $y=0.95$; and $x=20$, $y=1.55$. It will be seen that these values agree closely with the measured horsepower for these velocities. By making $x=0$ in equation 12 we have $y=-0.125$. The vertex of this maximum power curve is at a distance 0.125 below the axis of x. If the origin of coordinates be changed to this point, equation 12 will

take the form $x^2=K\,y'$, K being a constant and y' the horsepower referred to the new origin. Hence we see that the maximum horsepower varies as the square of the wind velocity.

To find the variation of the speed and load for this curve DK, we notice that the 3-pound curve is tangent at $x=10.5$, the 5-pound curve at $x=14$, the 8-pound curve at $x=19$, and the 11-pound curve at $x=24$. The horsepower is known at these points, so that the speed can be found from equation 7.

The wind velocity, load, horsepower, and speed for each of these points are as follows:

Data regarding points of tangency of power curves with curves of maximum power of Aermotor No. 27.

Wind velocity per hour.	Load.	Horsepower.	Revolutions per minute.
	Pounds.		
5 miles	0	0	0
10.5 miles	3	0.33	21
14 miles	5	0.70	26
19 miles	8	1.40	33
24 miles	11	2.17	37

The speeds here found are platted in fig. 42, giving the curve PQ, which gives the speed of the wheel for the maximum load. This curve is seen to be a nearly straight line for velocities above 9 or 10 miles an hour. Hence we may say that the speed increases as the first power of the wind velocity for maximum power.

The load for any wind velocity can now be found from the formula $\text{H. P.} = \frac{176\ Lu}{33{,}000}$, the speed being taken from the speed curve PQ. Or the loads can be measured from the load curve RS, fig. 42. This load curve (RS) for maximum power is seen to be a straight line, showing that for wind velocities above 9 or 10 miles an hour the load for maximum power varies nearly as the first power of the wind velocity.

The following table contains some interesting facts in regard to the working of this mill:

Data in regard to speed of mill No. 44—16-foot Aermotor.

Wind velocity per hour.	No load.			Maximum load.			Ratio of speed at maximum load to speed at no load.
	Revolutions per minute.	Circumference velocity in miles.	Ratio of circumference velocity to wind velocity.	Revolutions per minute.	Circumference velocity in miles.	Ratio of circumference velocity to wind velocity.	
8 miles	23	13.2	1.67	15	8.6	1.08	0.65
12 miles	38	21.7	1.81	23	13.2	1.10	0.61
16 miles	48	27.4	1.71	29	16.6	1.04	0.60
20 miles	56	32.0	1.60	34	19.4	0.97	0.60
25 miles	64	36.6	1.46	38	21.6	0.86	0.60

It will be seen that the ratio of the circumference velocity of the wheel to the wind velocity increases to 12 miles an hour and then decreases. In a 12-mile wind the circumference of the wheel is moving 1.81 times faster than the wind that drives it. It will be seen also that the circumference velocity of the wheel when carrying the maximum load is about equal to that of the wind that drives it, and that the speed of the wheel when carrying a maximum load is about 39 per cent less than its speed when carrying no useful load.

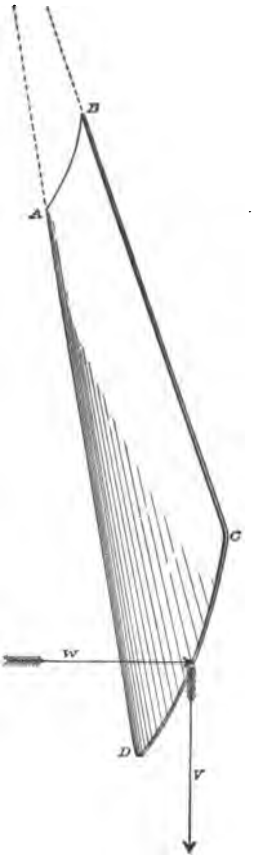

FIG. 58.—Sail of 16-foot Aermotor

ACTION OF AIR ON THE SAIL OF AN AERMOTOR.

It is not our purpose to discuss this action from a theoretical point of view, but to explain it from the observed and computed results of the 16-foot Aermotor. Fig. 58 shows the concave surface of one sail of a 16-foot Aermotor in a nearly horizontal position as it moves downward; w represents the velocity of the wind, and v the circumference velocity of the sail. The curve JAE, fig. 59, shows the outer end of the sail, and to, in fig. 60, shows the inner end. The cords of these arcs, or the plane of the sail, makes an angle (JOP) of 30° with the plane of the wheel. The point E, fig. 59, represents a particle of air as it comes in contact with the sail when the wheel is carrying its best load; EF represents the velocity of the wind, EH the velocity of this point of the sail; then EG, the other side of the parallelogram constructed on EH and EF, is the velocity of this particle of air over the sail; EG is not tangent to the sail. The point A represents a particle of air as it comes in contact with the moving sail, when the mill is carrying no load; AC represents the relative velocity of the particle of air. It will be seen that the air does not enter the sail tangent to it, but more nearly tangent for best load than for no load.

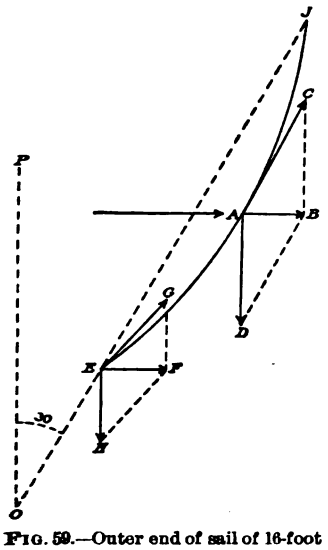

FIG. 59.—Outer end of sail of 16-foot Aermotor.

In fig. 60 t represents a particle of air as it strikes the inner end of the sail when the mill is carrying no load, and a represents a particle of air as it strikes the inner end of the sail when the mill is carrying its maximum load. It will be seen that the air does not strike the sail tangent to it at any place for any load, but that it is most nearly tangent at the outer end of the sail at maximum load. In order for the air to enter the sail tangent to it at maximum load, the angle POJ should be a little greater than 30° at the outer end, and considerably greater than 30° at the inner end. As the load is decreased the angle POJ should be decreased.

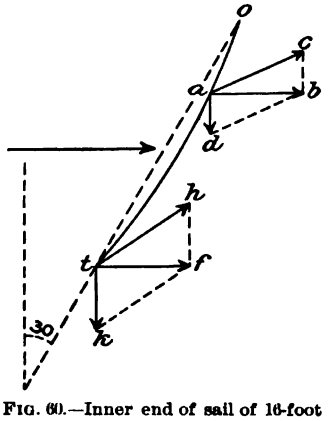

FIG. 60.—Inner end of sail of 16-foot Aermotor.

USEFUL WORK OF TWO POWER MILLS IN A GIVEN TIME.

We can find the useful work of the 12-foot and the 16-foot Aermotors in a year, as we have for two pumping mills (pp. 69–71). For this purpose we will use the mean wind movement at Dodge, Kansas, from 1889 to 1893, as given by Mr. Willis L. Moore, Chief of Weather Bureau.[1] The mean number of hours per month that the wind velocity was 0 to 5, 6 to 10, etc. miles an hour at this place is given in the following table, also the mean horsepower of these two mills for each month. The number of horsepower hours for each mill each month is given at the bottom of the table. The horsepower hours for any month are found by multiplying the number of hours by the horsepower and adding the products.

Table showing useful work of 12-foot and 16-foot Aermotors in a year.

Month.	Mean wind movement at Dodge, Kansas, 1889–1893.							Total hours.	Horsepower hours.	
	0 to 5 miles.	6 to 10 miles.	11 to 15 miles.	16 to 20 miles.	21 to 25 miles.	26 to 30 miles.	31+ miles		12-foot mill.	16-foot mill.
	Hrs.	*Hrs.*	*Hrs.*	*Hrs.*	*Hrs.*	*Hrs.*	*Hrs.*			
January	200.9	253.0	156.2	74.4	37.2	14.9	7.4	744	250.5	358.0
February	176.0	230.1	128.6	74.4	40.6	20.3	6.8	677	251.6	361.5
March	126.5	208.3	178.6	119.0	59.5	29.8	22.3	744	386.6	558.9
April	115.2	172.8	158.4	115.2	72.0	43.2	43.2	720	461.2	671.3
May	119.0	193.5	171.1	119.0	74.4	37.2	29.8	744	433.9	629.4
June	122.4	187.2	136.8	108.0	86.4	50.4	28.8	720	452.0	658.2
July	141.4	215.8	178.6	119.0	59.5	22.3	7.4	744	339.8	489.3
August	178.6	230.6	156.3	96.7	59.5	14.9	7.4	744	297.5	427.6
September	165.6	180.0	151.2	93.6	72.0	36.0	21.6	720	379.6	550.5
October	208.3	230.6	141.4	74.4	52.1	22.3	14.9	744	294.0	423.5
November	194.4	266.4	129.6	64.8	36.0	14.4	14.4	720	244.9	350.9
December	186.0	267.9	141.4	81.8	44.6	14.9	7.4	744	262.2	374.6
Mean									337.8	487.8
Horsepower 16-foot mill		0.13	0.56	1.25	2.00	3.15				
Horsepower 12-foot mill		0.10	0.41	0.85	1.36	2.12				

[1] Some Climatic Features of the Arid Region, by Willis L. Moore. Washington, 1896.

The work done by these mills is greatest at this place in April (461 horsepower hours for the 12-foot and 671 horsepower hours for the 16-foot) and least in November (245 horsepower hours for the 12-foot and 351 horsepower hours for the 16-foot). The mean monthly power is 338 for the 12-foot mill and 488 for the 16-foot mill. Stating these results in another way, we may say that the 12-foot mill at this place will furnish on an average 1.3 horsepower 10 hours a day for 26 days a month, and the 16-foot mill will furnish 1.9 horsepower 10 hours a day for 26 days a month. It must be remembered that the wind velocity on the Great Plains is considerably greater than in the eastern part of the United States, and that consequently the horsepower hours of these mills when used in New York State, for example, will be considerably less than those given in the foregoing table.

MATHEMATICAL DISCUSSION OF TESTS OF JUMBO MILL NO. 55.

On page 46 we have given the results of tests of a 15.5-foot Jumbo mill working two 6-inch pumps. In order more fully to determine the power of this mill and its variation with the number and size of the sails, we have had constructed mill No. 55, shown in Pl. XVI, *B*. It is made of wood, the parts being fastened together with bolts. The diameter is 7.75 feet; length of sails, 11½ feet. There are 8 sails, each made of two boards 11½ feet long and 8 inches wide. There is no governor or other method of regulating the speed of the mill at high wind velocities, as in other mills, but there is a large door or shield on each side. By opening these the mill may be stopped. The mill is not fastened to the ground, but may be moved around by hand so that the wind strikes the sails at right angles. The shaft is 4 by 4 inches and 14 feet long, carefully turned down in a lathe to a diameter of 3 inches near each end. The friction brake is of wood, has an arm about 3½ feet long, and is made so as to fit on the end of the shaft. Oil was freely used on the brake. It was not found practicable to use loads greater than about 6 pounds on a 35-inch arm, as the friction burned the shaft; but the results for the four loads used showed that up to the maximum load the speed of the wheel, or the number of revolutions per minute, varies directly as the load, so that we can easily compute the load and speed for maximum power in any wind velocity. The weight of the wheel with its 8 sails was about 450 pounds. The coefficient of axle friction for well-oiled yellow pine is probably about 0.10, so that the axle friction was about 45 pounds. This weight (45 pounds), acting with a 1½-inch arm, is equivalent to about 2 pounds applied on the brake with a 35-inch arm. The friction on starting is probably 50 to 100 per cent greater than the friction of motion. The 0 brake load then really corresponds to a brake load of 2 or more pounds.

Four sets of tests were made of this mill, numbered 1, 2, 3, and 4. In the first set the full sail area of 8 sails, each 11½ feet by 16 inches,

was used. The number of revolutions of the wheel for the four brake loads of 0, 1.75, 2.5, and 4.5 pounds was determined for velocities from 7 to 22 miles an hour. In the second set of tests there were 8 sails, each having an area of 11½ feet by 8 inches; that is, each sail was only half as wide as those used in the first set of tests. In the third set of tests the sail area consisted of 4 sails, each 11½ feet by 16 inches; that is, every other full sail was removed. The fourth set of tests was made to determine the effect of concentrating the air on the sails and reducing the resistance due to air striking the shield and glancing upward by the use of an inclined surface of approach to wheel. This inclined surface had a length of 7 feet and formed an angle of 11° with the horizontal.

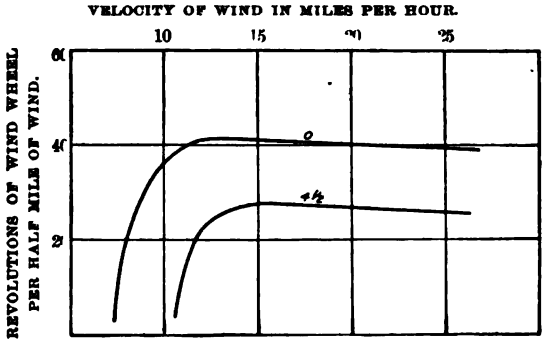

FIG. 61.—Diagram showing revolutions of wind wheel of mill No. 55—7.75-foot Jumbo. Curve marked 0 is for no brake load; curve marked 4½ is for a brake load of 4.5 pounds.

Test No. 1.—In Pl. XVI, *B*, the mill is shown with 8 sails, each 11½ feet by 16 inches. Fig. 61 shows the number of revolutions of the wheel per half mile of wind for two loads—0 and 4.5 pounds—on a 35-inch arm, for the wind velocities shown. These curves for a mill without any means of governing in high velocities are given for comparison with curves of mills having a governor. It will be seen that these curves are nearly horizontal straight lines beyond the point of maximum revolutions per half mile. Thus, for no brake load the revolutions at 12 miles are about 62, and at 25 miles about 59. In a mill with a governor, as, for instance, that shown in fig. 36, the curve is more inclined, or the drop in the number of revolutions is greater.

Fig. 62 shows the number of revolutions per minute for several loads, fig. 63 shows the horsepower.

The results of these tests are as follows:

Results of tests of Jumbo mill No. 55 with 8 sails 11½ feet by 16 inches.

Load on brake.	Load per revolution of wind wheel.	Revolutions of wind wheel per minute at given wind velocities (per hour).					Horsepower at given wind velocities (per hour).				
		8 miles.	12 miles.	16 miles.	20 miles.	25 miles.	8 miles.	12 miles.	16 miles.	20 miles.	25 miles.
Lbs.	*Ft.-lbs.*										
0	0	11	24	33	41	49	.....	.....	.....	.....	.....
1.75	32.0	.....	22	30	38	45	.....	0.022	0.030	0.036	0.043
2.5	45.8	.....	20	28	36	43	.....	0.028	0.039	0.050	0.060
4.5	82.5	.....	17	25	33	40	.....	0.043	0.063	0.082	0.100

A. VIEW OF MILL NO. 57—24-FOOT LITTLE GIANT.

B. VIEW OF MILL NO. 55—7½-FOOT JUMBO.

From these results it is seen that the reduction in the number of revolutions per minute is proportional to the load. For example, in a 20-mile wind a $4\frac{1}{2}$-pound load reduces the number of revolutions from 41 to 34, or 1.75 revolutions per brake pound. The power $= 2\pi\ RnL \div 33{,}000$, where $n =$ the number of revolutions of the wheel per minute, $L =$ the load on brake, in pounds, $R =$ the arm of brake (35 inches), $\pi = 3.1416$, and $33{,}000 =$ the number of foot-pounds per minute in a horsepower. We may write the power thus: $P = KnL$, and compute its value as follows, K being a constant equal to $2\pi R \div 33{,}000$:

$$
\begin{aligned}
P_0 &= K \times 41 \quad \text{(revolutions)} \times 0 \ (L) = 0 \\
P_1 &= K \times 33 \quad \text{(revolutions)} \times 4.5 \ (L) = 148.5K. \\
P_2 &= K \times 26 \quad \text{(revolutions)} \times 8.5 \ (L) = 221.0K. \\
P_3 &= K \times 22.5 \quad \text{(revolutions)} \times 10.5 \ (L) = 236.3K. \\
P_4 &= K \times 20.75 \quad \text{(revolutions)} \times 11.4 \ (L) = 238.6K. \\
P_5 &= K \times 19 \quad \text{(revolutions)} \times 12.5 \ (L) = 237.5K.
\end{aligned}
$$

P_4 is seen to be larger than any of the other values of P, and gives an approximate value of the power of the mill for that wind velocity (20 miles).

We may find a more accurate value in an easier way. Let x be the load for maximum horsepower in any given wind velocity. We have seen that 1 pound of load reduces the speed by 1.75 revolutions. Then we can write $P = K\ (41 - 1.75x)\ (x)$. For a maximum value of P we must differentiate P with respect to x, place the first differential coefficient 0, and solve for x. This value of x, according to calculus, makes the power a maximum. Differentiating, we have $P \div dx = 41 - 2\ (1.75x) = 0$. Solving for x we have $x = 11.7$ pounds. The corresponding value of the revolutions of the wheel per minute is $41 - 1.75 \times 11.7 = 20.5$ revolutions.

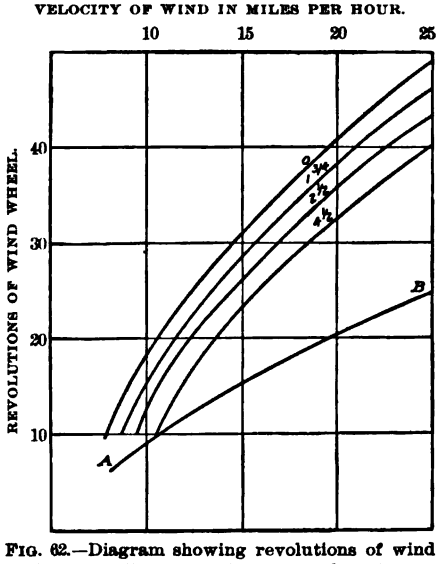

FIG. 62.—Diagram showing revolutions of wind wheel of mill No. 55—7.75-foot Jumbo. Curves marked 0, $1\frac{1}{4}$, $2\frac{1}{4}$, and $4\frac{1}{4}$ are for brake loads of 0, 1.75, 2.5, and 4.5 pounds, respectively; AB is for best load.

For a 25-mile wind we have $x = 49 \div 3.5 = 14$ pounds, and $n = 16.5$. In a similar way we get the load and revolutions for maximum power

for other wind velocities. These and the corresponding horsepowers are as follows:

Values for the curve DK (maximum power) fig. 63.

Wind velocity (miles per hour).	Load in pounds.	Revolutions of wind wheel per minute.	Horsepower.
8	3.1	6.0	0.010
12	7.0	12.0	0.046
16	9.4	16.5	0.086
20	11.7	20.5	0.133
25	14.0	24.5	0.190

The curve *DK* is nearly a parabola. The revolutions per minute for the best load are seen to be very nearly equal to the wind velocity

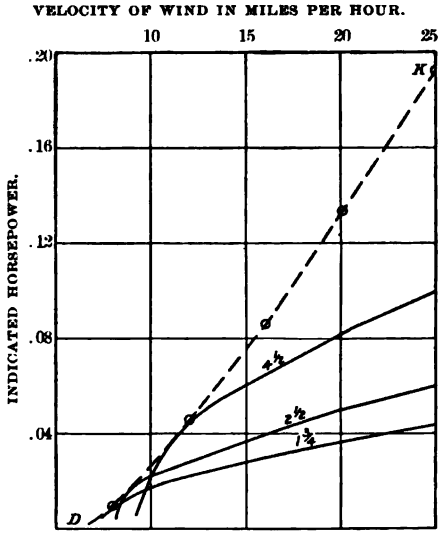

FIG. 63.—Diagram showing horsepower of mill No. 55—7.75-foot Jumbo. The curves show the horsepower for brake loads of 1.75, 2.5, and 4.5 pounds, respectively; dotted curve *DK* shows maximum power.

in miles. These are platted in fig. 62, giving the line *AB*, which is nearly straight.

RELATION BETWEEN WIND VELOCITY AND CIRCUMFERENCE VELOCITY OF WHEEL.

If we multiply the number of revolutions of the wind wheel for any wind velocity by 24.4 feet (the circumference of the wheel) and divide by the wind velocity, in feet per second, we have the following results for no brake load and for best load:

Table showing ratio between wind velocity and circumference velocity of wheel.

Load.	Wind velocity per hour.				
	8 miles.	12 miles.	16 miles.	20 miles.	25 miles.
No load	0.38	0.55	0.57	0.57	0.55
Best load	0.21	0.27	0.28	0.28	0.27

From this we see that the velocity of the circumference of the wheel is not more than 57 per cent of the velocity of the wind. For velocities above a certain amount it remains nearly constant for any load. It will be seen too that the speed of the wheel for best load is almost exactly half that for no brake load.

Test No. 2.—Sail area, 11½ feet by 8 inches—each inner half sail removed. The results of this set of tests are as follows:

Results of tests of Jumbo mill No. 55, with 8 sails 11½ feet by 8 inches.

Load.		Revolutions of wind wheel per minute at given wind velocities (per hour).					Horsepower at given wind velocities (per hour).				
		8 miles.	12 miles.	16 miles.	20 miles.	25 miles.	8 miles.	12 miles.	16 miles.	20 miles.	25 miles.
Pounds.	*Ft.-lbs.*										
0	0	9	22	30	39	47	.....	.....	.....	.....	.....
3.25	59.6	.....	17	26	32	40	.....	0.031	0.047	0.058	0.072

By comparing these results with those of test No. 1 it will be seen that the number of revolutions per minute for no load is from two to three times less when the half sails are used. The 3½-pound load with half sails gives about the same speed as the 4½-pound load with whole sails. The weight of the wheel is reduced about 40 per cent, which makes the reduction in speed less than it would be if the weight of the wheel remained constant. It will be seen, then, that for this size of wheel very little power is gained by the use of the inner 8-inch board of each sail. It is quite likely that sails 12 inches wide would give fully as much power as sails 16 inches wide.

Test No. 3.—The sail area was 4 sails, each 11½ feet by 16 inches—every other full sail removed. The results of this set of tests were almost the same as those of test No. 1. There was no measurable reduction in the speed of the wheel when every other full sail was removed. The weight of the wheel was reduced about 40 per cent, and consequently the friction. The gain in pressure on the extra sail area is counterbalanced by the additional friction.

Test No. 4.—The sail area was 8 sails, each 11½ feet by 16 inches—the same as for test No. 1. There was an inclined surface (shown in Pl. XVI, *B*) for concentrating air on sails and in a measure prevent-

ing an upward current from the front shield. The results of this set of tests are the same as those of No. 1. There was no measurable increase in the number of revolutions when the mill was loaded or unloaded, or when the incline was used or not used.

For this size of mill 4 sails each 12 inches wide give the maximum power. From our tests of other mills we should say that the sail width should increase directly as the diameter increases. For diameters of 12 feet or more it is likely that the addition of one or two more sails, say 5 for a 12-foot mill and 6 for a 16-foot mill, may increase the power over that for 4 sails.

In 1895 we made some measurements of the pressure of air on small curved surfaces,[1] from which we infer that if galvanized-iron sails curved to a radius about twice the width and with the concave surface to the wind were used the power of the mill would be increased about 15 per cent over that with the plane fans.

Putting the results of these tests of Jumbo mills in the most practical form, we have the following as the proper sail area and the probable horsepower of wooden Jumbo mills in a 16-mile wind when properly loaded, assuming the power to increase as the square of the diameter:

Table showing proper sail area and probable horsepower of Jumbo wooden mills in a 16-mile wind.

Diameter of wheel.	Number of sails.	Width of sails.	Length of sails.	Horsepower.
		Inches.	*Feet.*	
8 feet	4	12	12	0.09
12 feet	5	18	12	0.20
16 feet	6	24	12	0.36
20 feet	6	30	12	0.56

The formula used for computing the pressure on a series of plane surfaces moving in the direction of the velocity of the wind is—

$$P=F\frac{r}{g}(c-v)^2 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (a)$$

In this F is the area, in square feet, of the vane, r the heaviness of air at the observed temperature and barometric pressure, c the velocity of wind, and v the velocity of wind wheel, each in feet per second.

The heaviness of the air is found from the formula—

$$r=ro\frac{B}{Bo}\frac{To}{T} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (b)$$

In this To is the absolute temperature in centigrade degrees.

[1] Kansas University Quarterly, Vol. IV, July, 1895.

From equation b we have $r=0.08\times\frac{29}{30}\times\frac{273}{301}=0.07$ pound per square foot. From the table on page 123 it will be seen that for maximum power in a 16-mile wind $v=0.28c$. Substituting in equation a we have, for the pressure on one sail—

$$P=11.5\times\frac{1}{32.2}\times0.07\,(1-0.27)^2\,c^2=4.1 \text{ pounds.}$$

The arm of this pressure about the axis of the wheel is about 3.06 feet. Hence the moment of this force is $4.1\times3.6=14.76$ foot-pounds. This moment is equal to the moment of the brake load, and we have $14.76=35x$, and $x=4.24$ pounds. The load actually carried on the brake, including friction, is about 8 pounds; hence nearly half of the working pressure comes from wind pressure on the approaching and receding sails, or only a little more than half the pressure comes from the sail which is at the highest position possible.

MATHEMATICAL DISCUSSION OF TESTS OF LITTLE GIANT MILL NO. 56.

This is a 4.67-foot mill made by Mr. C. Hunt, of Wichita, Kansas. These mills are made in sizes from 4 to 24 feet in diameter, to rest on a low tower or on a building. The largest one yet built is shown in Pl. XVI, *A*. It is used for grinding wheat. The Little Giant mill will be seen to resemble the Jumbo in that the wind wheel moves in the direction of the wind and not across it. It differs from the Jumbo, however, in having a vertical axis and many curved iron sails, instead of a horizontal axis and few plane wooden sails. The wind is prevented from striking the sails as they come around toward the wind by a shield which, when closed, covers about one-third of the circumference. The shield can move freely about the axis of the mill and has hinged to it a wing which can be held at right angles to the circumference. There is also a vane fastened to the shield, to aid in the government of the mill. When the wing of the shield is closed, the vane takes the direction of the wind and places the shield directly in front of the wind wheel, shutting off the wind from the wheel. When the wing is open, the pressure of the wind against it carries the shield around, admitting the wind to one-half of the wheel. By properly placing the vane and using the proper weight on the wing, the wind is admitted to a small or a large portion of the wheel, and thus the speed of the wheel is regulated. The mill receives the wind from all directions and regulates automatically.

Mill No. 56 has 24 curved iron sails, each 3 feet 10½ inches long and 6½ inches wide, set at an angle of 27° to the radius. The radius of curvature of the sails is 7½ inches. The vertical shaft of the wind wheel has a beveled cogwheel, which gears into another beveled cogwheel on a short horizontal shaft. The latter has a pitman for working a pump. The horizontal shaft on which the brake was placed

was geared back 43÷13. The brake arm was 2 feet 10½ inches long. The number of revolutions per minute of the brake shaft was found for the four loads 0, 2, 4, and 6 pounds, respectively. The corresponding speed of the wind wheel is found by multiplying by 3¼. The results of the tests were as follows:

Results of tests of Little Giant mill No. 56.

Load.		Revolutions of brake shaft per minute at given wind velocities (per hour).					Horsepower at given wind velocities (per hour).				
		8 miles.	12 miles.	16 miles.	20 miles.	25 miles.	8 miles.	12 miles.	16 miles.	20 miles.	25 miles.
Pounds.	*Ft.-lbs.*										
0	0	10	19	25	31	38	.....	.....	.....	.....	.....
2	35.7	6	16	21	27	34	0.007	0.017	0.023	0.030	0.040
4	71.4	.....	11	18	23	30	.....	0.024	0.039	0.050	0.065
6	107.1	.....	6	14	19	26	.....	0.019	0.045	0.062	0.084

Referring to fig. 64, the mill will be seen to start for no load in a light wind—about 5 miles an hour. More than half of each speed curve is a nearly straight line. This is due to failure to govern, the mill being held wide open all of the time. It will be seen that each pound of load reduces the speed about two revolutions per minute for about 0.67 of each curve. Fig. 65 shows the brake horsepower for three loads—2, 4, and 6 pounds, respectively. The curve DK, to which these load curves are tangent, shows the maximum power of the mill. This curve passes through the points y (horsepower)=0 when x (wind velocity)=5, and y=0.024 when x=12. Hence, if we assume this curve DK to be a parabola, we have for its equation—

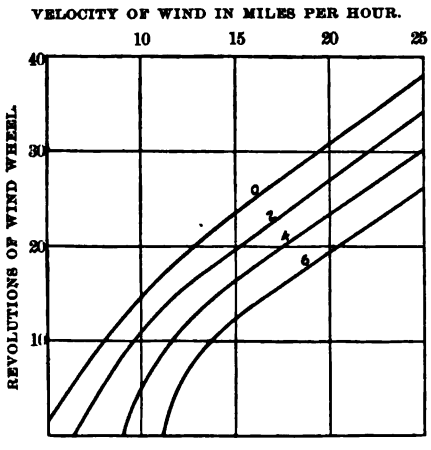

FIG. 64.—Diagram showing revolutions of wind wheel of mill No. 56—4.67-foot Little Giant. The curves marked 0, 2, 4, and 6 are for brake loads of 0, 2, 4, and 6 pounds, respectively.

$$x^2 = 496y + 25.$$

The horsepower from this equation and from fig. 65 is as follows, y' being taken from fig. 65:

Horsepower of mill No. 56 at given wind velocities.

x	y	y'
5	0.000	0.000
8	0.008	0.007
12	0.024	0.024
16	0.046	0.045
20	0.075	0.065

It will be seen that the horsepower of this mill does not increase as fast as the square of the wind velocity. From fig. 65 it will be seen that the 2-pound curve is tangent at 8 miles, the 4-pound curve at 12 miles, and the 6-pound curve at 16 miles an hour. Hence the load for maximum power increases about as the first power of the wind velocity. The speed of the brake shaft for these loads is 6, 11, and 14 revolutions per minute, respectively. Hence the speed of the wind wheel does not increase as fast as the first power of the wind velocity. The ratios of circumference velocity of wind wheel to wind velocity for no load and for maximum load for five wind velocities are as follows:

Table showing relation of circumference velocity of wind wheel to wind velocity.

Wind velocity per hour.	No load.			Maximum load.			Ratio of revolutions at maximum load to revolutions at no load.
	Revolutions of brake shaft per minute.	Circumference velocity in miles per hour.	Ratio of circumference velocity to wind velocity.	Revolutions of brake shaft per minute.	Circumference velocity in miles per hour	Ratio of circumference velocity to wind velocity.	
8 miles	10	5.2	0.65	6	3.1	0.39	0.60
12 miles	19	9.9	0.74	11	5.7	0.47	0.58
16 miles	25	13.0	0.81	14	7.3	0.46	0.56
20 miles	31	16.1	0.80	16	8.3	0.42	0.52
25 miles	38	19.8	0.79	.....	.....	.....	.....

From this we see that for no load the greatest circumference velocity is only 81 per cent of the wind velocity. For best load the ratio is 47 per cent. Here is the great disadvantage of these wheels, which move in the direction of the wind instead of across it—they move too slowly. The ratio of circumference velocity to wind velocity in an Aermotor is 1.75. This is 2.16 times the greatest corresponding circumference velocity of the Little Giant mill.

COMPARISON OF LITTLE GIANT AND JUMBO MILLS.

From the following table it will be seen that the circumference velocity of the Jumbo is only from 0.55 to 0.70 of that of the Little Giant for best load. The horsepower of the Jumbo is about 1.9 times that of the Little Giant; the ratio of the Jumbo diameters is 1.67;

hence the ratio of the power is a little greater than that of the diameters. It must be remembered, however, that the Jumbo is about three times the length of the Little Giant. For the same sail lengths the latter does 1.57 more work than the Jumbo.

Comparative data of Little Giant and Jumbo mills.

Mill.	Maximum horsepower at given wind velocities (per hour).					Circumference velocity for maximum load at given wind velocities (per hour).				
	8 miles.	12 miles.	16 miles.	20 miles.	25 miles.	8 miles.	12 miles.	16 miles.	20 miles.	25 miles.
7½-foot Jumbo	0.010	0.046	0.086	0.133	0.190	1.7	3.40	4.70	5.80	.....
4½-foot Little Giant	0.007	0.024	0.045	0.066	0.100	3.1	5.70	7.30	8.30	.....
Ratio	1.4	1.9	1.9	2.0	1.9	0.55	0.60	0.64	0.70	.....

Taking into account the difference in the diameters of the wind wheels of these mills, we may say that the Little Giant will furnish about 2.5 more power than the Jumbo for the same diameter and length of sail. The Jumbo requires a 7-mile wind to start it with no load; the Little Giant will start with no load in less than a 5-mile wind. The Jumbo has no means of governing; the Little Giant governs easily and completely. The Jumbo gets the full pressure of the wind when it comes from two directions only; the Little Giant works equally well with the wind from any direction. The Little Giant is less likely to be injured in a windstorm than the Jumbo. The first cost of the Jumbo is somewhat less than that of the Little Giant. A 5-foot Little Giant with stub tower can be bought for about $15.

COMPARISON OF LITTLE GIANT WITH 8-FOOT AERMOTOR.

The efficiency of the pump and well of Aermotor No. 5 (for description and results of tests, see pages 33 and 34, Part I) is probably about 60 per cent. For the speeds and horsepowers of these mills we have the following:

Comparative data of Little Giant and 8-foot Aermotor.

Mill.	Load per revolution of wind wheel.	Sail area.	Revolutions of wind wheel per minute at given wind velocities (per hour).				Horsepower at given wind velocities (per hour).			
			12 miles.	16 miles.	20 miles.	25 miles.	12 miles.	16 miles.	20 miles.	25 miles.
	Ft.-lbs.	*Sq. ft.*								
8-foot Aermotor (No. 5)	45	34	62	84	99	115	0.084	0.115	0.138	0.158
4½-foot Little Giant (No. 56)	24	54	34	55	71	92	0.024	0.039	0.050	0.065
Ratio	1.96	0.63	1.82	1.53	1.40	1.25	3.50	3.00	2.76	2.43

It will be seen that the sail area of the Aermotor is only 0.63 that of the Little Giant, that the wind wheel of the former makes from 1.25 to 1.82 more revolutions per minute than the latter, and that the power of the former is from 3.5 to 2.4 times greater than the power of the latter. While the sail area of the Aermotor is only 0.63 that of the Little Giant, the wind area of the former is much greater than that of the latter. The wind can not enter the wheel of the Little Giant over an area greater than the radius multiplied by the length of the sail, or 9.1 square feet. In the Aermotor the wind enters the wheel over an area equal to the difference between the areas of the two circles, one having a diameter of 8 feet, the other having a diameter of 3 feet. This wind area is 43.2 square feet. Hence the wind area of the Aermotor is $43.2 \div 9.1 =$ 4.75 times that of the Little Giant. Here is the great advantage that the Aermotor has over the Little Giant—it has only 0.63 of the sail area, and hence cost less for sails, and 4.75 times more air strikes its sails than strikes those of the Little Giant. It will be shown later that in the Little Giant the air acts on its sails while passing out of the wind wheel as well as while passing into the wheel, and thus the power for the same wind area is greater in that mill than in the Aermotor.

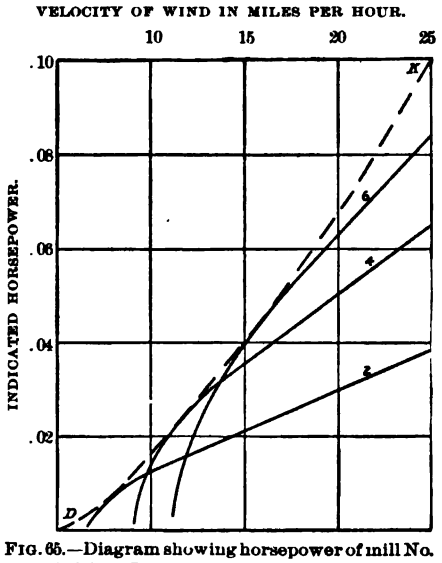

FIG. 65.—Diagram showing horsepower of mill No. 56—4.67-foot Little Giant. The curves marked 2, 4, and 6 show horsepower for brake loads of 2, 4, and 6 pounds, respectively; dotted curve *DK* shows maximum power.

Fig. 66 is a diagram showing the action of the wind on the sails of the Little Giant mill. *AH*, *BK*, etc., are the curved sails. The cord *AH* makes an angle of 27° with the radius *AP*. *CD* is the shield, with the wing *DE* open. Let *Aa* represent the direction and magnitude of the wind with respect to the earth. We have seen that for maximum load the ratio of the circumference velocity of the wheel to the wind velocity is 0.47; hence drawing *Ac* tangent to the circumference *Aa* and equal to 0.47 of *Aa*, and completing the parallelogram on them, we have *Ab* representing the direction of the wind with respect to the moving sail. If we assume this to be the velocity of the air over the sail (it is somewhat less than this, since *Ab* is not quite tangent to the sail at entrance), we can construct the path of a particle of air with respect to the earth. The points 1, 2, and 3 are on this path, and *L3* gives the approximate direction of the particle of air through the wheel. At *L* we combine this velocity with the inner circumference velocity

of the sail, giving the velocity *Lp* with respect to the moving sail. This is seen to make a large angle with the tangent at entrance, reducing its magnitude somewhat. *L4* is approximately the absolute path of the particle over the sail, and 4-5 is the direction of the particle at exit. *B6* represents the absolute path of a particle of air as it moves over the sail *BK*. This particle of air passes through the

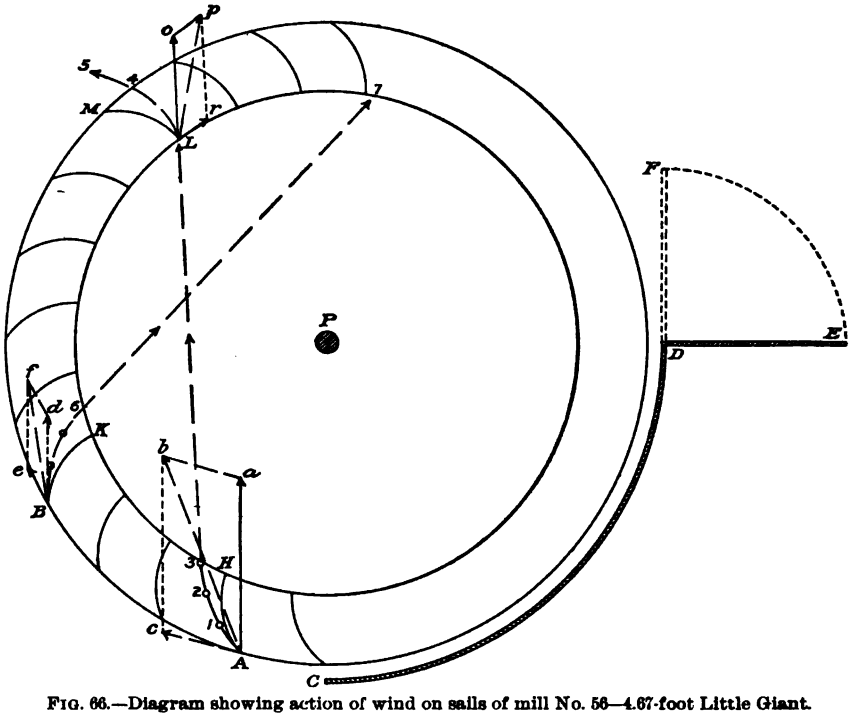

FIG. 66.—Diagram showing action of wind on sails of mill No. 56—4.67-foot Little Giant.

wheel in the direction 6-7. The path 6-7 of this particle crosses the path of the particle from 3; hence there is interference inside the wheel, which prevents our tracing with accuracy the path of a particle out of the wheel. It is evident, however, that after passing into the wheel the air strikes the concave side of the sails on the opposite side, and aids in pushing the wheel around, so all the work is not done by the sails on the side where the air enters.

INDICATED AND TRUE VELOCITIES.

Thus far all of our results for speed and power of windmills are given in terms of indicated velocities, that is, velocities as read from the Robinson cup anemometer. It is necessary, or at least desirable, to examine these to see whether they agree with true velocities or distances actually passed over by the wind per hour. Fig. 67 shows the Robinson cup anemometer as used by the United States Weather Bureau. This instrument was invented in 1846 by Dr. T. R. Robinson, of Armagh, Ireland, and is now used by several meteorological bureaus for the measurement of wind velocity. It gives a continuous record of wind movement and requires no device, such as a vane, to give it the proper direction with respect to the wind. It is made so that each 50 revolutions of the cups can be read on the dial, and there is an electrical device for recording each 250 or each 500 revolutions of the cups.

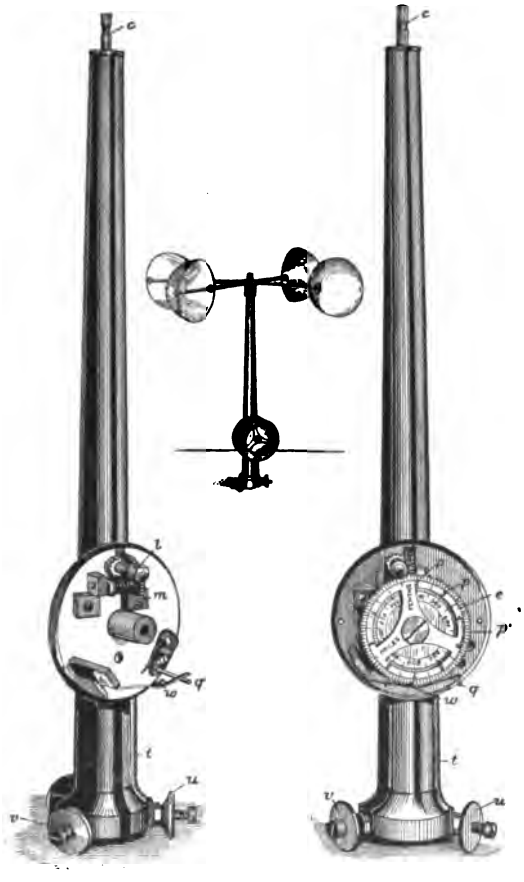

FIG. 67.—Anemometer and cups. c, spindle which forms the axis of revolution of the cups; m, 50-tooth wheel which engages an endless screw on the end of the spindle; l, small toothed wheel which engages an endless screw on the axis of the wheel m; e, pair of dial wheels which are moved by the wheel l; p, one of ten contact pins to aid in closing the electric circuit at the end of each mile of wind; p', two of these pins connected, forming the tenth mill pin; w, contact spring; q, a contact point at the end of contact spring; t, small insulated tube connecting q with the insulated binding post u and with the second binding post v.

Referring to fig. 68, let A = the upper and B = the lower cup of a Robinson anemometer rotating about the axis, let c equal the velocity of the wind, and v the velocity of the cup center, each in feet per second; x = the ratio of the velocity of the wind to that of the cup center, P_1 = the pressure on the concave surface, and P_2 = the pressure on the convex surface of the cup. Dr. Robinson found from sixteen experiments with stationary cups exposed to wind of several velocities that for all velocities the pressure when the concave surface of

the cup is toward the wind is about four times that when the convex surface is presented to the wind. The pressure on the moving cup A is $P_1=K_1 Fr(c-v)^2 \div 2g$,[1] and the pressure on the cup B is $P_2=K_2 Fr(c+v)^2 \div 2g$; F being the area of the cup, K_1 and K_2 being constants the ratio of which, as found by Robinson, is 4, r the heaviness of air, and $c+v$ and $c-v$ the relative velocities of the cups. Neglecting friction in the anemometer, inertia of cups and arms, and the influence of two of the cups until they are near the position shown in fig. 68, we see that for uniform velocity P_1 must equal P_2; if P_1 is greater than P_2, v will increase; if P_1 is less than P_2, v will decrease. We have, therefore, for uniform velocity $K_1 Fr(c-v)^2 \div 2g = K_2 Fr(v+c)^2 \div 2g$ or $4(c-v)^2 = (c+v)^2$. This equation can be put in the form $4(x-1)^2=(x+1)^2$. Solving we have $x=3$; that is, the velocity of the wind is three times that of the cup centers. For an anemometer having arms 6.72 inches long, the distance passed over by a cup center in 500 revolutions is $d=\frac{2\times 22}{7}\times\frac{6.72}{12}\times 3\times 500=5{,}280$ feet, or 1 mile.

Dr. Robinson believed that this ratio of wind velocity to that of cup center was true for all velocities, and consequently the makers of the instrument have marked the dial in miles. We shall see, however, from the rating of the one that we have used in these windmill tests that this ratio is not a constant. It will be seen that for wind of uniform velocity this ratio is a variable which has the value 3 for 9 miles an hour, is greater than 3 for less velocities, and is less than 3 for greater velocities than 9 miles an hour; in other words, to get true velocities we must add a correction below 9 miles an hour and subtract a correction above that velocity. Friction and inertia were neglected in deriving the foregoing value of this ratio. The former has little influence in an instrument kept in good condition, but in a poorly kept instrument it may have a large influence for low velocities. This ratio has been found to be 8 to 10 with much friction. The inertia of the arms and cups has a marked influence on this ratio, especially for ordinary gusty wind. As the

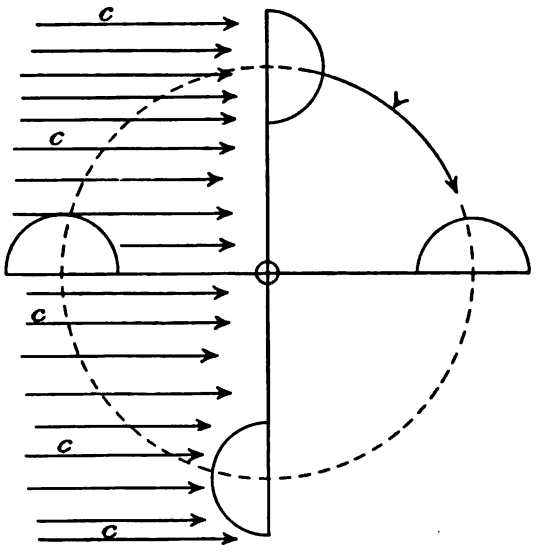

FIG. 68.—Diagrammatic section of anemometer cups.

[1] Church's Mechanics of Engineering, p. 815.

gustiness of the wind increases the correction to be subtracted to get the true velocity increases also.

The relation between the indicated and true velocity of an anemometer is found by moving the anemometer in still air at different velocities, and noting the distance passed over, also the readings of the instrument. It is seldom that the air out of doors is still for any considerable length of time, so that this comparison is usually made within an inclosure, the anemometer being carried around in a circle. The radius of the whirler should be as long as possible, and made so as to affect the circulation of air as little as possible, and to reduce the effect of the centrifugal force.

The whirling machine that we have used for rating the anemometer is the property of the United States Weather Bureau. It consists essentially of an arm 28 feet long and 8 feet above the ground, on the end of which the anemometer is carried at an elevation of 2 feet above the arm. This arm is counterweighted and is stiffened by the rods. It is clamped to a vertical shaft, which carries a cogwheel near its lower end. A cogwheel on a horizontal shaft engages the large cogwheel and gives rotation to the arm. For low velocities the power was applied through a crank on the horizontal shaft, and for higher velocities by a crank on a second shaft, the latter working the first shaft by means of two sprocket wheels and a chain. The machine was set up out of doors, in a sheltered place away from any building, and was used on several nights when there was scarcely any wind. It was made to rotate about half the time in the positive direction and the other half in the negative direction.

A new Robinson anemometer was used with which to compare results obtained for the one used in our windmill tests. The results for these instruments agree quite closely. The results obtained on December 29, when there was no perceptible wind, are as follows:

Table showing relation between indicated and true velocity of anemometer.

Indicated velocity, in miles per hour.	Revolutions of long arm in ¼ mile of wind.	True velocity, in miles per hour.	Correction, in miles.	Correction, in miles, for gusty wind.
6	15.5	6.20	+0.20	0
8	15.1	8.05	+0.05	−0.2
11	14.7	10.78	−0.22	−0.6
15	14.2	14.20	−0.80	−1.2
20	13.8	18.40	−1.60	−2.2
25	13.53	22.55	−2.45	−3.2
30	13.3	26.60	−3.40	−4.3

It will be seen that the indicated velocity is less than the true velocity for velocities less than 9 miles an hour; above 9 miles the true velocity is less than the indicated velocity. In other words, the correction is added below 9 miles an hour and subtracted above that velocity. It will be seen that up to 11 miles an hour these

anemometer readings differ very little from the true readings, but for higher velocities the correction becomes quite large. The last column of the foregoing table gives the corrections to be applied to the indicated velocities of the Weather Bureau Robinson anemometer for gusty wind.[1] The motion of ordinary moving air, when studied with a very light anemometer recording each revolution, is found to vary suddenly by large amounts. The rate of motion changes 20 or 30 miles an hour in a few seconds.[2] The record of the standard Robinson anemometer, recording miles or half miles, does not show these sudden changes, but gives an average velocity for the wind. Its

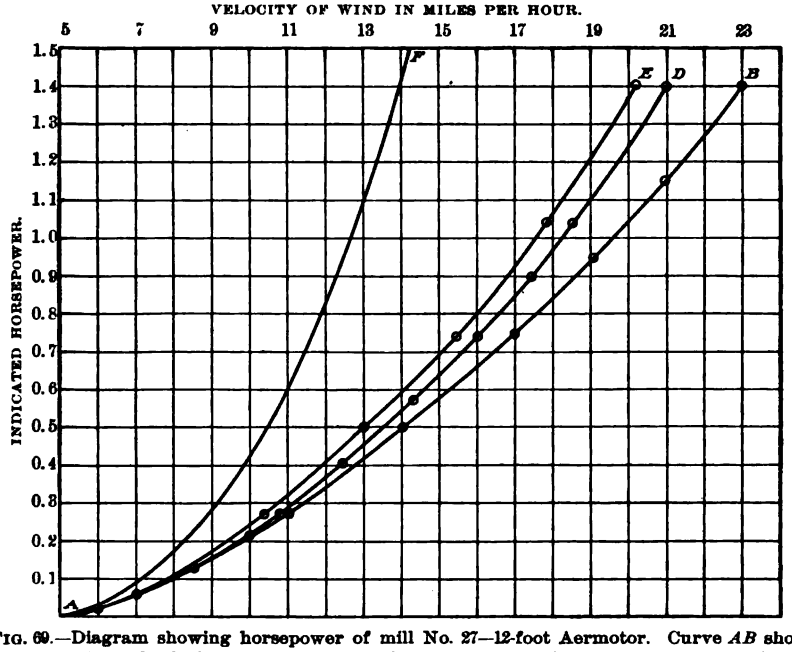

FIG. 69.—Diagram showing horsepower of mill No. 27—12-foot Aermotor. Curve *AB* shows power for indicated velocity, assuming wind to be not gusty; *AD* shows power for true velocity; *AE* shows power in average gusty wind; *AF* shows power from H. P. $= \frac{x^3 - 140.6}{2,086}$.

weight and consequent inertia cause it to continue its rotation for a time after the impulse is passed, and when the next impulse strikes the cups their weight will not allow them to take the velocity of the impulse. The less the weight the more nearly will the velocity of the cups be that of each gust. The effect is that the cups revolve faster in a gusty wind of, say, 20 miles an hour, than in wind of the same velocity but not gusty. Two difficulties arise in dealing with gusty wind: (1) The gustiness of any wind varies from time to time, and the anemometer gives no indication of it; (2) we have no means of producing artificial gusty wind in which to rate anemometers. If the anemometer is rated in natural wind, then a correction

[1] Anemometry, by C. F. Marvin. Washington, 1893.
[2] The Internal Work of the Wind, by S. P. Langley. Washington, 1893.

must be applied, but its amount is uncertain. The velocity of gusty wind can not, therefore, be measured with certainty. This, however, does not lessen the value of the Robinson anemometer as an instrument for measuring wind velocity, nor introduce an error in our results. For a given velocity and gustiness the anemometer will always give the same reading, and the same gusty wind which strikes the anemometer strikes the windmill directly behind it. The difficulty arises when we try to compare results in which the Robinson anemometer was used in measuring velocity with those in which some other form of anemometer was used. The corrections given in the last column of the table were found by Prof. C. F. Marvin, of the United States Weather Bureau, and are for ordinary gusty wind.

The Robinson anemometer is now so generally used to measure wind velocity that it is better to express the speeds and powers of windmills in terms of these than of any other. If results are desired in terms of true velocities, they can be found approximately by means of the table of corrections given on page 133. Fig. 69 shows the power of mill No. 27 expressed in terms of three kinds of velocity: *AB* shows the power assuming the wind to be not gusty; *AD* shows the power for true velocity, that is, after applying the corrections in the fourth column of the table; *AE* shows the power in average gusty wind, that is, after applying the corrections in the last column of the table; *AF* would give its power if the power increased as the cube of the indicated velocity.

COMPARISON OF WRITER'S EXPERIMENTS WITH THOSE OF OTHERS.

Comparison with Smeaton's experiments.—In this comparison we use indicated velocities and results for best mills. Smeaton's results are given on pages 15 and 16, Part I. It will be remembered that his wheels were 3.5 feet in diameter, moved against still air in a circle of 5.5 feet radius, and that his wind velocities varied from 3 to 6 miles an hour.

Smeaton found (maxim 1, page 15, Part I) that the velocity of a windmill sail, whether loaded, so as to produce maximum power, or unloaded, is nearly as the velocity of the wind. We have found that the velocity of wind wheel when loaded increases nearly as the wind velocity, but when it is unloaded it increases as the square root of the wind velocity.

Smeaton found (maxim 3) that the maximum power increases somewhat less rapidly than as the cube of the wind velocity. We have found that the maximum power increases as the square of the wind velocity—for true velocities somewhat faster than as the square of the wind velocity.

Smeaton found (last part of maxim 5) that the power for a constant load increases as the first power of the wind velocity. Our experiments show that for a constant load the power increases as the square root of the wind velocity.

Smeaton found (maxim 6) that the circumference velocities of similarly made mills of different diameters vary inversely as the diameters. We have found this to be true.

Smeaton found (maxim 8) that the maximum power for similarly made mills increases as the squares of the diameters. We have found that it increases about as the 1.25 power of the diameters.

Smeaton found (maxim 9) that the circumference velocity of the Dutch sail, whether loaded or unloaded, is considerably greater than the wind velocity. We find that it is nearly equal to the wind velocity for loaded sail, and about 1.75 times the wind velocity for the unloaded sail.

Comparison with Coulomb's experiments.—In this comparison indicated velocities are used. Coulomb's observations (see page 16, Part I) were made on a Dutch mill (fig. 1, Part I) having a wind wheel 70 feet in diameter. He found that at a wind velocity of about 15 miles an hour the wind wheel was making 13 revolutions per minute and yielding about 7 horsepower of useful work. Comparing this with the results for the 16-foot Aermotor for maximum load, we have the following:

Comparison of results of tests of Coulomb's 70-foot mill with writer's 16-foot Aermotor.

Mill.	Revolutions of wind wheel per minute.	Circumference velocity in feet per minute.	Horsepower.
70-foot mill	13	2,860	7.0
16-foot Aermotor	28	1,408	0.8

The circumference velocity of the large mill is more than twice that of the smaller one. The ratio of the powers is $7 \div 0.8 = 8.8$. The ratio of the diameters is $70 \div 16 = 4.4$. The ratio of the powers is about twice that of the diameters. The ratio of the squares of the diameters is 19.14.

It is very likely that the wind velocity as found by Coulomb is too small; the very large circumference velocity of his wheel indicates this. It is probable that the wind velocity during his observations was about 20 miles an hour instead of 15. In a 20-mile wind the horsepower of the 16-foot mill is about twice that in a 15-mile wind, and the speed 34 revolutions per minute against 28. The ratio of the horsepowers would then be about as the diameters of the wind wheels.

Comparison with Griffiths's experiment.—The performance of a pumping windmill depends on so many factors, most of which may affect the result to a large degree, that it is doubtful whether it is worth while to make a comparison where the conditions differ much. There is the pump efficiency alone, other conditions being the same,

which may make the horsepower of one four or more times that of the other. For example: Both mills have a brake power of 1 horsepower; one mill works a pump which has an efficiency, under present conditions, of 20 per cent, its useful horsepower being 0.2; the other mill operates a pump which has an efficiency of 80 per cent, its horsepower being 0.8, or four times that of the other. We have seen that for wind velocities above a certain amount the power increases nearly as the load, so that by doubling the load for the higher velocities the power is doubled. The gearing and means of governing affect the power in a somewhat less degree. The way in which the wind velocity is measured may affect the recorded power and speed. If the wind wheel, or tower, or any other obstacle obstructs the free flow of the air to the anemometer, the recorded velocity will not be as great as it should be. If the anemometer is placed on the platform, it will give a less velocity than if held some distance in front of the wheel and at the height of the axis. The temperature and barometric pressure affect the power.

In Mr. Griffiths's data (see pages 17 and 18, Part I) the load factor is known, but the wind velocities he gives are small, and for small velocities it is difficult to compute the effect on the power of differences in load. In fact, most of his velocities are less than are required to start irrigating mills. Only four of his mills, viz, Nos. 1, 2, 5, and 6, are comparable with mills that we have tested. The mills with which we have compared them are Nos. 36, 38, and 47—very lightly loaded mills with low pump efficiency.

Comparison of results of Griffiths's tests with those of writer's tests.

Mill.	Outer diameter of sail.	Load per stroke of pump.	Wind velocity.	Strokes of pump per minute.	Horsepower.
	Feet.	*Ft.-lbs.*	*Miles.*		
Griffiths's No. 1	22.3	480.0	7.0	6.8	0.098
Writer's No. 36	22.5	248.0	7.0	5.0	0.038
Griffiths's No. 2	11.5	29.2	5.8	18.0	0.011
Writer's No. 38	10.0	21.0	5.8	9.0	0.006
Griffiths's No. 5	10.2	51.0	8.5	20.5	0.028
Writer's No. 38	10.0	21.0	8.5	21.0	0.014
Griffiths's No. 6	9.8	30.7	6.0	12.5	0.012
Writer's No. 47	10.0	37.0	6.0	7.0	0.008

It will be seen from this table that the horsepowers of the mills tested by Mr. Griffiths are greater than the horsepowers of the mills we have tested. The speeds of the wheels are also greater, except in Nos. 5 and 38. We are of the opinion that a part of the difference in results is due to difference in the method of measuring the wind velocity. The wind velocity found by Mr. Griffiths is less than we have found it.

Comparison with King's experiments.—Professor King's measurements of the brake horsepower of a 16-foot Aermotor are given on

page 20, Part I. These results are plotted in fig. 70, giving the curve *AB*. The curve of maximum power for the 16-foot Aermotor No. 44 is redrawn in this figure as curve *CD*. It will be seen that these curves are nearly parallel to 16 or 17 miles an hour, and then diverge rapidly. Believing that much of this difference is due to the wind wheel of Professor King's mill interfering with the anemometer, we have experimented with two anemometers, one located 29 feet directly south of the wind wheel of mill No. 44, the other located 27 feet directly north of it, and each recording, side by side, the wind velocity on an electric register. The hourly wind velocity for fourteen consecutive hours, during the first ten of which the mill was working and during the last four of which it was out of the wind, is given in the following table:

Table showing wind velocities during writer's experiments with anemometers on mill No. 44.

Hour.	Direction of wind.	Wind velocities.		Ratio.
		Front anemometer.	Rear anemometer.	
		Miles.	*Miles.*	*Per cent.*
First	SE	15.0	10.2	0.68
Second	do	18.0	13.4	0.74
Third	do	18.0	13.5	0.75
Fourth	do	19.5	12.8	0.65
Fifth	SW	18.0	13.5	0.74
Sixth	do	16.8	12.6	0.77
Seventh	S	15.2	10.0	0.65
Eighth	SE	12.7	9.6	0.76
Ninth	do	10.5	7.5	0.71
Tenth	do	7.7	5.5	0.72
Eleventh	do	7.2	7.2	1.00
Twelfth	do	8.6	8.6	1.00
Thirteenth	do	10.5	10.6	1.00
Fourteenth	do	10.3	10.4	1.00

The direction of the wind is the mean for each hour, as shown by an anemoscope. When the wind was from the southeast it occasionally came for a short time almost directly from the east; and, again, when from the southwest it occasionally came for a time from the west.

It will be seen that the hourly velocity, 27 feet, behind the running wheel was only 65 to 77 per cent of that in front of the wheel. As soon as the wheel was turned out of the wind the two anemometers recorded nearly the same velocity.

Referring now to the direction of the wind when Professor King's windmill tests were made, and remembering that his anemometer was 40 feet directly east of the moving wheel, it will be seen that nearly all of the tests were made when the wind came from the northwest or

the southwest, and that consequently the moving wheel must have interfered with the proper working of the anemometer, causing it to record a less velocity than actually existed, and making the horsepower greater than if the anemometer had been in front of the wheel. In fig. 70 it will be seen that an increase of from 5 to 25 per cent in Professor King's wind velocities would move his curve over to the right of our curve. These curves are found in very different ways—Professor King's from 26 single observations, ours from more than 150 observations. Instead of finding points on this curve when the proper load is unknown, we have found speed and power curves for constant loads, and from these drawn the curve of maximum power. None of the mills that we have tested have given a power curve like *AB* in fig. 70. It will be noticed too that the curve *CD* is quite like the corresponding one for the 12-foot Aermotor. This we should expect, since the mills are similar in construction.

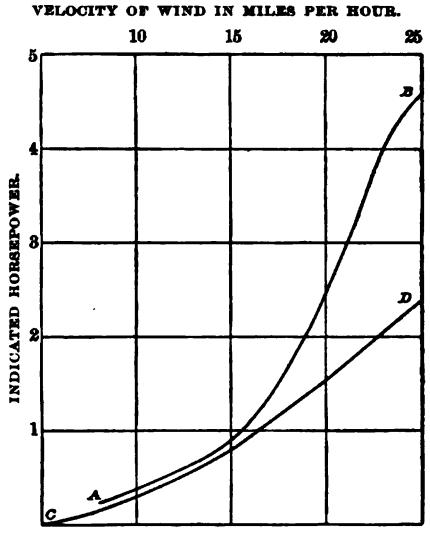

FIG. 70.—Diagram showing horsepower of two 16-foot Aermotors. Curve *AB* shows brake horsepower of Professor King's 16-foot Aermotor; *CD* shows maximum power of writer's 16-foot Aermotor No. 44.

Comparison with Perry's experiments.—Indicated velocities are used in this comparison. Some of Mr. Perry's results are given on pages 20 and 21, Part I. His tests were made on wheels 5 feet in diameter, carried against still air in a circle 14 feet in diameter, and his wind velocities were not greater than about 11 miles an hour. Mr. Perry states that his results agree with those of Smeaton. A comparison of our results with Smeaton's has been given on pages 135 and 136.

We will compare in detail the working of two of Mr. Perry's wheels, viz, Nos. 44 and 48, with that of our 12-foot Aermotor No. 27.

Mr. Perry's wheel No. 44 is somewhat like the wind wheel of our 12-foot Aermotor No. 27. It has 12 curved wooden sails, each 18 by 12.3 by 5.8 inches, having a weather angle at the inner end of sail of 30° and an angle of 25° at the outer end. The air is more obstructed in its passage through this wheel than through the wheel of No. 27. The following are some of the results for these mills in an 8.5-mile wind, the only velocity which Mr. Perry gives for his mill.

Comparison of results of tests of Perry's 5-foot mill No. 44 with writer's tests of 12-foot Aermotor No. 27.

Mill.	Area of sail.	Wind velocity.	Load per revolution.	Maximum load.		Horse-power.	No load.	
				Revolutions per minute.	Circumference velocity.		Revolutions per minute.	Circumference velocity.
	Sq. ft.	*Miles.*	*Ft.-lbs.*					
Perry's mill No. 44	13.6	8.5	11.3	44.4	8.6	0.016	84.3	22.1
12-foot Aermotor No. 27	73.0	8.5	22.2	19.0	11.9	0.13	32.5	20.4
Ratios	5.3	.....	19.6	.....	1.38	8.12	.....	0.92

It will be seen that the load, in foot-pounds, per revolution of wind wheel is 19.6 times greater and the circumference velocity for maximum load is 1.38 times greater for No. 27 than for the 5-foot wheel, but that the circumference velocity for no load is a little less for No. 27 than for the other mill. The power of the 12-foot Aermotor is more than 8 times that of the 5-foot mill, and its sail area is 5.3 times greater.

We will next compare the 12-foot Aermotor No. 27 with Mr. Perry's 5-foot mill No. 48, which gave the greatest power of the 61 wheels tested by him. It had six curved pasteboard sails, each 19 by 23.7 by 10.9 inches, set at a weather angle of 35° at the inner end of the sail, and at an angle of 25° at the outer end. All obstructions to the free flow of air over the back of the sails were removed. The following are some of the results for these mills at a wind velocity of 11 miles an hour:

Comparison of results of tests of Perry's 5-foot mill No. 48 with writer's tests of 12-foot Aermotor No. 27.

Mill.	Area of sail.	Wind velocity.	Load per revolution.	Maximum load.		Horse-power.	No load.	
				Revolutions per minute.	Circumference velocity.		Revolutions per minute.	Circumference velocity.
	Sq. ft.	*Miles.*	*Ft.-lbs.*					
Perry's mill No. 48	14.2	11	23.3	66.8	17.4	0.047	142	37.2
12-foot Aermotor No. 27	73.0	11	333	28.5	17.9	0.288	45	28.3
Ratios	5.2	.....	14.3	.....	1.03	5.96	.....	0.76

It will be seen that the circumference velocities of these wheels for maximum power are nearly equal, but for no load the circumference velocity of the 5-foot mill is about 25 per cent greater than that of the 12-foot Aermotor. The power of the latter mill is nearly six times that of the 5-foot mill. There are more air obstructions in the wheel of the 12-foot mill than in that of the 5-foot mill, so that the difference in the power would be greater for equal air obstructions. For a cor-

responding amount of obstruction the ratio of power would probably be 6.5 to 7. This ratio is greater even than $12 \div 5^2 = 5.76$, the ratio of the squares of the diameters.

From this comparison of the results of our tests with those of Smeaton and Perry it will be seen that the power of a natural moving air of a given measured velocity is greater than the resistance of the air to a wheel carried around in a circle. Some of the laws (see page 114) which have been found to govern wheels moved against still air—notably that the power increases as the cube of the wind velocity—are not applicable to windmills in moving air.

ECONOMIC CONSIDERATIONS.

The power of windmills has been computed from tests on model windmills, in artificial air of low velocity, assuming, first, that the power increases as the cube of the wind velocity, and, second, that the power increases as the square of the diameter. Our tests of windmills recorded in the preceding pages show that the power does not increase much faster than as the square of the wind velocity, and about as 1.25 times the power of the diameter of the wind wheel. We believe that to these two false assumptions is due the exaggerated power of windmills claimed by windmill makers and others interested. A good 12-foot steel mill should furnish 1 horsepower in a 20-mile wind (indicated) and 1.4 horsepower in a 25-mile wind. This is the smallest amount of power that will do any considerable amount of useful work. A 16-foot mill will furnish 1.5 horsepower in a 20-mile wind (indicated) and 2.3 horsepower in a 25-mile wind.

A 12-foot steel mill and a 50-foot steel tower as commonly made weigh about 2,000 pounds. A 16-foot steel mill and a 50-foot steel tower weigh about 4,250 pounds. The 16-foot outfit weighs more than twice that of the 12 foot, and its power is only 1.5 that of the latter. In addition, the 12-foot mill will govern more easily and is less likely to be injured in a storm than the 16-foot mill. In most cases, therefore, it is better to use two 12-foot mills than one 16-foot mill.

The economic value of a windmill depends on its first cost, on the cost of repairs, and on its power. Most of the effort put forth at the present time to improve windmills is directed toward reducing the first cost. Competition is so strong that the cost must be kept low, and this is often accomplished at the sacrifice of the other two factors—cost of repairs and power. The pumping mills and their towers are, as a rule, too light and lacking in stiffness. It is said that in some parts of the West wooden mills are coming into use again, on account of the lightness and poor quality of the steel mills. This, however, is a fault of the making, not of the material. The wooden tower is stiffer and more rigid than the steel tower.

Power is the most important factor, and next to that should come strength, stiffness, and durability.

It has been shown that the steel mills, with their few large sails, have much more power than the wooden mills with their many small sails. (See page 106.) A mill should have as few moving parts as possible, in order that the loss of power by friction shall be small, also the liability to get out of working order be reduced to a minimum. The power of a mill is at best so small that if there is much friction there is little power left to do useful work. The grinder should be on the foot gear and not worked by a belt, and the shafting and cog-wheels should not be too heavy. In the large wooden mills the shafting is much too heavy; apparently it is designed on the assumption that the mill will furnish several times more power than it really can. The mill should be carefully erected, the vertical shafting exactly vertical and the horizontal shafting truly horizontal, so that there will be no binding of the parts. Poor workmanship is an important cause of the small power of some mills. Only a skilled workman who understands the business should be employed to erect a windmill.

The mills should be placed at a proper height above surrounding obstructions—at least 30 feet above the highest trees and buildings. This calls for a tower from 50 to 70 feet high. It is better to use a small wheel on a high tower than a large wheel on a low tower. An 8-foot wheel on a 70-foot tower will probably do more work in a given time than a 12-foot wheel on a 30-foot tower with trees and buildings around it. The tower should be firm and rigid, no shaking under a heavy wheel load. Steel towers are in constant vibration under heavy loads.

A mill should govern readily at the proper wind velocity, but this velocity need not be less that 30 miles an hour. A weight appears to be better than a spring for holding the wind wheel in the wind. The tension of a spring can not readily be changed when desired but may gradually lose its tension. (See pages 60 and 110.) There is very great need of an automatic device for changing the load on a pumping mill as the wind velocity changes. The mill should start in a light wind, say 4 to 5 miles an hour, or it will be idle many hours when it should be at work; but in order to do this it must be lightly loaded. In the higher wind velocities, with a light load the mill will do only a small fraction of the work it would do with a much heavier load. The increase in the load should be nearly proportional to the increase in the wind velocity. (See page 113.) Until such a device is invented the load should depend on the wind velocity of the place where the mill is to be used and on the amount of storage.

The pumping mill is ordinarily constructed so that all of the useful work is done on the upstroke of the pump, producing a jerky motion and excessive strain on the working parts. This defect is partly remedied by the use of a large plunger rod, which will force up some of the water on the downstroke. A second remedy is the use of a

lever with a heavy weight at one end, the other end being attached to the plunger rod. As the plunger moves down the weight on the end of the lever is raised on the upstroke. The descent of the weight assists the mill in lifting the water. Neither device is satisfactory. A pumping mill working direct stroke makes too many strokes per minute at wind velocities above about 15 miles an hour. The valves ordinarily used for small pumps will not work well if the number of strokes is greater than 30 per minute. The mill should be geared back about 2 to 1 for large mills and about 3 or 4 to 1 for small mills.

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56TH CONGRESS, 2d Session. | HOUSE OF REPRESENTATIVES. | DOCUMENT No. 486.

DEPARTMENT OF THE INTERIOR

WATER-SUPPLY

AND

IRRIGATION PAPERS

OF THE

UNITED STATES GEOLOGICAL SURVEY

No. 43

CONVEYANCE OF WATER IN IRRIGATION CANALS, FLUMES, AND PIPES.—FORTIER

WASHINGTON
GOVERNMENT PRINTING OFFICE
1901

UNITED STATES GEOLOGICAL SURVEY
CHARLES D. WALCOTT, DIRECTOR

CONVEYANCE OF WATER

IN

IRRIGATION CANALS, FLUMES, AND PIPES

BY

SAMUEL FORTIER

WASHINGTON
GOVERNMENT PRINTING OFFICE
1901

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LETTER OF TRANSMITTAL.

DEPARTMENT OF THE INTERIOR,
UNITED STATES GEOLOGICAL SURVEY,
DIVISION OF HYDROGRAPHY,
Washington, December 17, 1900.

SIR: I have the honor to transmit herewith a manuscript prepared by Prof. Samuel Fortier, of Bozeman, Montana, with the recommendation that it be printed in the series of Water-Supply and Irrigation Papers. The author has had much experience in the construction of irrigation canals and related hydraulic works used in the reclamation of the arid lands. The results of his experience and observation are of value in considering improved methods of utilizing the water resources of that section of the United States. It is important, therefore, to bring these details to the attention of engineers and others engaged in construction work.

Very respectfully,

F. H. NEWELL,
Hydrographer in Charge.

Hon. CHARLES D. WALCOTT,
Director United States Geological Survey.

CONVEYANCE OF WATER IN IRRIGATION CANALS, FLUMES, AND PIPES.

By Samuel Fortier.

INTRODUCTION.

The vast extent of arid land in the United States—fully two-fifths of its area—includes some of the richest agricultural land on the globe. Its utilization rests not merely upon obtaining water, but to a large extent upon bringing water to the land at a cost commensurate with the value of the crops raised. The question of cost, therefore, enters largely into all considerations of the extent to which the arid region can be redeemed by irrigation. With improved methods and appliances the cost of irrigating works can be greatly reduced and the area of tillable land correspondingly increased. This paper has been prepared for the purpose of calling attention to present practices in the conveyance of water in irrigation canals, flumes, and pipes, and to point out ways in which works of this character can be built with greater permanence and at less cost than those now in existence.

IRRIGATION CANALS.

The network of ditches and canals which pervades the cultivated portion of arid America varies in size from the furrow of the irrigator to the large canal of the corporation. While the capacity of the smaller may be less than 1 second-foot, that of the larger may exceed 1,000 second-feet. As a rule the small ditches were built by individual settlers, those of medium size by communities of farmers, and the large canals by capitalists.

LOCATION.

When a canal is to be built to convey water to a tract of land, considerable preliminary work is necessary before its location can be determined. It is not difficult to ascertain the total area of land under any proposed canal, but care and good judgment must be exercised in estimating the percentage of the total area which is arable and irrigable.

The acreage that can be irrigated having been determined, it is necessary to ascertain the volume of water that will be required for the tract—i. e., the net volume after deducting loss due to evaporation and seepage from the volume admitted through the head gates. The preliminary line may start from the source of supply or from the land to be watered. When the boundaries of the tract are known and there is little choice as to the location of the head gates, the better way is to start at the highest point of land to be watered and run toward the source of supply on a proper grade. On the other hand, if there is a particular location on the river from which water can be readily and cheaply diverted, it is well to begin at that point and extend the location, allowing for the necessary grade toward the land to be watered. This mode of procedure is preferable when the territory covered by the proposed canal is extensive and unsettled, and when economy in construction is of greater importance than the area to be served. The zero of the final location should be at the point of diversion.

In locating the center line of a canal the practice of the writer has been to employ two level parties. One level party precedes the transit party and sets temporary stakes on grade, irrespective of the sharpness of the bends or the irregularity of the general alignment. The other party follows the transit party and takes the elevations of the located line. Before proceeding to work the level party ascertains, from standard cross sections of the proposed canal, the depth of the cut at the center. The levelman, with his rodman, chainman, and stake-driver, places a long, slim stake, such as a lath, at each 100-foot station on surface grade. This surface is always at a fixed distance above the bottom of the excavated canal. If the proposed canal were to be 10 feet wide on the bottom, the difference between the bottom grade and the surface grade might be 2 feet; if the proposed canal were to be 16 feet wide, the difference might be 4 feet, and so on, in like proportion. The transit party lays down as straight and direct a location line as is possible, consistent with the grade stakes. These grade stakes are visible to the transitman between transit points, and represent the irregularities of the surface across which he is sighting. When their position is zigzag he usually can locate a straight line or a curve between them, so as to have an equal number on each side of the line and thus equalize, in a measure, the excess and deficiency of excavation. A strict adherence to the grade line makes a bad location. It not only increases the length of the line, but introduces a multitude of sharp curves, which are a serious objection in the operation of canals. The transitman may also lessen the distance by building low embankments at the head of dry ravines. Instead of following the grade stakes from *A* around to *C* and *B*, fig. 1, he may extend the line from *A* to *B*, and build a small embankment on the low side of the located

line. In canals of low grade such ponds as *C*, *B*, *A* make good settling basins, where the silt accumulates instead of in the main channel.

STANDARD CROSS SECTIONS.

When the preliminary lines are being run, notes are taken of the slope of the ground and the character of the materials. From this information, coupled with a knowledge of the required capacity, standard cross sections are made to suit the various slopes and capacities.

In these sections the material excavated is usually placed in such a manner as to form the embankments. On level ground the embankments are equal, on sloping ground the greater part of the excavated material is deposited on the low side, and on steep slopes all of the material excavated goes to form the lower embankment.

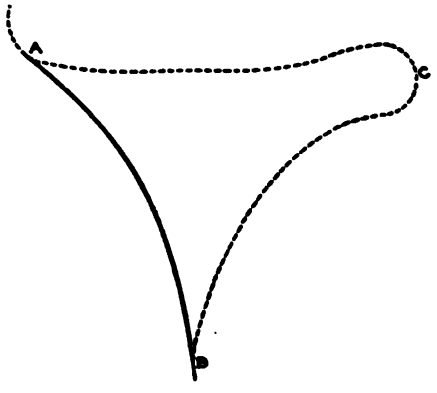


FIG. 1.—Embankment on low side of a canal.

In fig. 2 is shown a half section of a canal on level ground to carry about 100 second-feet of water with a mean velocity of between 2½ and 3 feet per second. It is 12 feet wide on the bottom, with side slopes of 1 to 1, a berm of 3 feet, and embankments 6 feet wide on top. The depth of water is 3 feet. To the right is a half section of the same canal, showing the form it is likely to assume after being in use several years. In comparing this half section with the former half section it will be noted that the center of the bed is slightly eroded, the sharp corner of the berm worn down, and the material deposited along the bottom edge. The embankment also has settled and is of a semielliptical form.

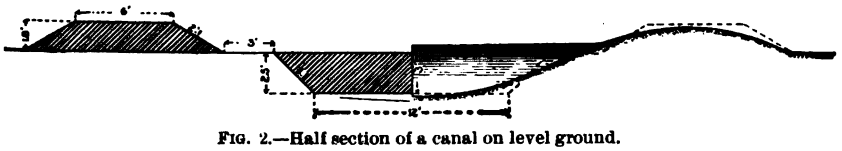


FIG. 2.—Half section of a canal on level ground.

From an examination of many sections obtained from canals long in use the author is led to believe that few channels in earth remain long in the form in which they were first constructed. The materials most commonly met with assume, in time, a cross section similar to that shown in fig. 2. Such being the case, the utility of berms is apparent. In the case under consideration a part of the excavated material would find its way into the channel and have to be removed a second time. In deep cuts or on steep hillsides berms are dispensed with, for

the reason that it is more economical to remove once a year the material which rolls down than to enlarge the section sufficiently to introduce them.

The top width of the embankments can not be made much less than 6 feet, unless some kind of large grader is used to remove the dirt. When teams are employed they crowd, unless the driveway is nearly 6 feet wide. The top of the embankment should also be of a uniform grade and parallel to the bottom grade. Any excess of materials should be placed on the outer slope, and not piled up in heaps above the grade of the embankment.

In locating a canal it is difficult to plan for just enough earth to make the embankments—in almost every case there is either a surplus or a deficiency. If the standard cut at the center, or grade rod, as it is termed, is 2½ feet, as in fig. 2, there will be portions with a depth of 2 feet or less and other portions with a depth of 3 feet or more. If these are adjacent, the excess in the one may suffice for the shortage in the other. There are times, however, when borrowing from some other source must be resorted to. Some prefer to borrow from the bed of the canal; but this seems to be a bad practice. By scooping out the bottom in places the grade and mean velocity are no longer uniform, and it is difficult to repair or clean the canal, on account of the pools of stagnant water which are formed in such pits. Others prefer to widen the canal without digging below grade. There are serious objections to these sudden changes of cross section, but on the whole the practice is better than interfering with the grade. Perhaps the better way is to plan the standard cross sections so that the amount excavated will be in excess of that required to form the embankments. When a low place in the location is encountered, the excess material can be hauled a distance of 200 feet or more from each side, to make up the deficiency. When the excavated material is more than sufficient to form the embankments, the surplus can be added to the outer slope without detracting from the appearance of the canal.

GRADES.

It is difficult to determine aright the proper grade or fall for a proposed canal. It is true that the velocity of water depends primarily on the grade, but there are so many conditions which affect the flow, that, having fixed upon a fall in a given length of canal, it is not easy to predict what the mean velocity will be. This mean velocity is always important, for in nearly every canal in earth there is a rate of motion which is adapted to the character of the materials through which the water flows. If this motion be too great, the finer particles will be carried away, leaving a mass of porous materials through which much water will seep and percolate, thus reducing the volume in the canal. On the other hand, if the motion of the water be too slow, the conse-

quences are also serious. The sediment borne by the swift currents of mountain streams is deposited in the artificial channel and would in time fill it if not periodically removed. The capacity of the canal, owing to deficiency in velocity, is also reduced, and, as is pointed out elsewhere, aquatic plants seem to grow more abundantly and give much more trouble in canals having a low velocity. As already stated, velocity depends on the volume carried, on the form of cross section, and on other factors. The velocity of water in a farmer's lateral on a grade of 50 feet to the mile may not be so great as that of the large canal on a grade of 1 foot to the mile. One of the laterals of the Logan and Hyde Park canal, in Cache County, Utah, has a fall of 52 feet to the mile, and when measured had a mean velocity of 1⅛ feet per second and a discharge of 0.85 second-foot. The Point Lookout canal, in Boxelder County, Utah, on a grade of 1.43 feet to the mile, has a mean velocity of 1.48 feet per second and a discharge of 87¼ second-feet. This great difference in grade, while the velocities are nearly equal, is to be accounted for in both the forms of the channels and the volumes of water carried. In the small lateral the ratio between the water area and the frictional surface was 1 to 4, while in the large canal this ratio was less than 1 to ½.

For ditches and canals built in common earth the mean velocity should not vary much from 2¾ feet per second under a full head. The behavior of such watercourses in the West during the last fifteen years seems to verify this statement. Assuming this to be a fact, it only remains to allow sufficient grade to produce, under given conditions, the required mean velocity.

The injurious effects of either too much or too little grade are well shown on two canals in Gallatin County, Montana. The Middle Creek canal was built about thirty years ago, to divert water from Middle Creek. The grade was established by means of a triangle and plumb line. With the dread ever present in the minds of the operators of getting too little fall, they went to the other extreme and allowed too much fall. The grade near the upper end is nearly 1 per cent, but it diminishes to about 1 in 200 over the fourth mile, while the main branches are on the slope of the valley, which is from 75 to 80 feet to the mile. In consequence, the velocity near the head is, in places, more than 5 feet per second, and the bed of the channel is washed clean of all earth and sediment, leaving a porous mass of coarse gravel and cobbles. So great is the seepage through this formation that the loss in 4 miles was 21.5 second-feet, or 22 per cent of the total flow during the month of July last. The effect of high velocities on the Fowler Switch canal, California, is shown in Pl. I, *A*.

The Kleinschmidt canal, Montana, was built about twelve years ago to divert water from West Gallatin River. It was well located, by a competent engineer, but the grade adopted was only sufficient to pro-

duce, when running full, a mean velocity of about 2 feet per second. Had this canal been operated to its full capacity during the last ten years, the effects of the low grade might not have been apparent; but inasmuch as the volumes carried have varied from 25 to 50 second-feet, instead of 100 second-feet, its approximate maximum capacity, the deposition of silt has been very great. In places this silt is found spread over the bottom, but more frequently there is only a thin layer on the bottom and the balance is found fairly regularly deposited on the edges, as represented in fig. 3. If one may judge by present conditions, the bottom width was originally 14 feet, with side slopes of 1 to 1, a berm on the lower side, and the embankment about 4 feet 6 inches above grade. This supposed original section is shown by the dotted line, while the heavy line, with the filling of silt at the edges, represents existing conditions. The grade of this canal, with the exception of the upper 7,000 feet, is 2.64 feet to the mile. While the mean velocity of the present flow is not known, it is presumed to be

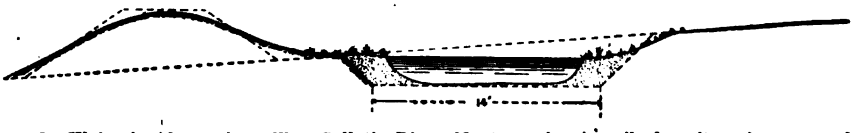


FIG. 3.—Kleinschmidt canal, on West Gallatin River, Montana, showing silt deposit on bottom and edges.

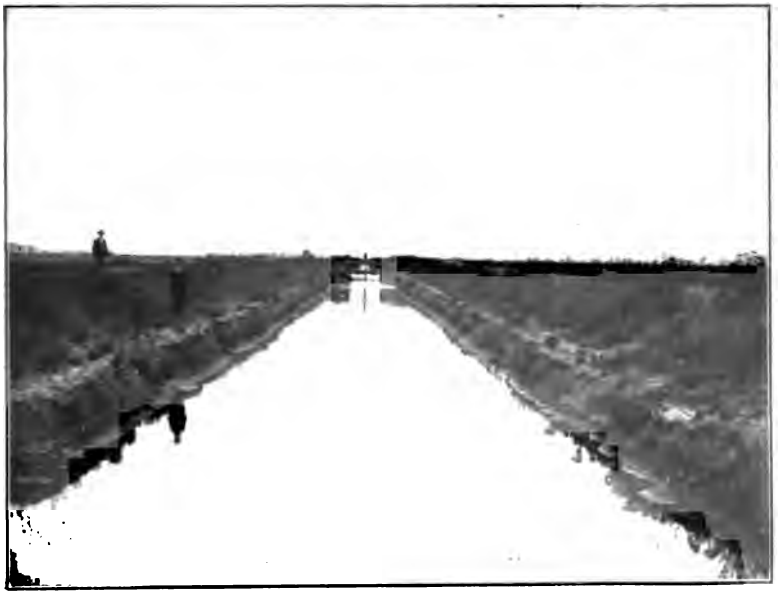
not more than 1½ feet per second; the calculated velocity, when the depth is 2½ feet, is a trifle more than 2 feet per second.

OPERATING CANALS IN WINTER.

The irrigation canal is frequently the only source of water supply of the new settler. In many unirrigated localities in the West well water is often difficult, if not impossible, to obtain. When extensive areas of land are irrigated every summer for a number of years, a part of the water so used is absorbed by the earth and gradually fills the underlying strata. In this way the water level is brought nearer the surface of the ground, and water can often be obtained by digging inexpensive wells. Until this change is effected, however, the new settler depends on the canal, not only to irrigate his crops, but to furnish water for his home and his stock. Hence, for the accommodation of farmers who have no other means of obtaining water, irrigation canals are frequently operated throughout the year, with the exception of a short period in the early spring when the water is turned out to facilitate cleaning and repairs.

When the water conveyed in the canal performs the dual function of irrigating land and furnishing a domestic supply for a town or city, it is necessary that its efficiency be maintained during the winter months. The small cities of the West, with populations varying from 1,000 to 6,000, have usually a small waterworks plant and the beginning of a

A. FOWLER SWITCH CANAL, CALIFORNIA, SHOWING EFFECT OF HIGH VELOCITIES.



B. BEAR RIVER CANAL, UTAH, LOOKING NORTH.

sewer system. Often their available means will not permit the construction of a conduit to tap the waters of a natural stream, in which case a temporary arrangement is made by extending the distributive system of the water mains to the nearest available canal until such time as means can be provided to carry the supply pipe to the river. This forms the second class of canals that are operated during the winter months.

The third class is operated solely for mechanical purposes, and since power is needed for milling, manufacturing, and mining in winter as well as in summer, efforts are made to keep such canals full of water during even the coldest weather. The difficulties which canal superintendents and mill men experience in doing this consist not in the removal of the surface ice, but in getting rid of and overcoming the bad effects of anchor ice and slush ice. The conduits and canals of the Rocky Mountain region head for the most part in the canyons adjacent to the cultivated valleys. In winter the temperature within these canyons is from 10° to 30° F. lower than that of the bordering plain, which fact, together with the exposed positions and the high velocity of the water, seems to be favorable to the formation of so-called anchor ice. This form of ice differs from surface ice in that it adheres to the bottom and sides of the exposed channel in small, long crystals until the size of the water channel is greatly reduced and in time becomes entirely choked. As the weather grows warmer the ice which has formed beneath the surface is detached, rises to the surface, and, as slush ice, floats down with the current. This slush ice not infrequently grounds in the bed of the channel and other ice accumulates at the same place, until an ice dam is formed which causes a break in the canal bank. The portion which floats down the canal is likely to choke the intake of the waterworks or prevent the operation of the gates and water wheels of the mills.

Usually ice forms in thin sheets on the surface of cold water. The average weight of a cubic foot of clear river water is about 62½ pounds; the weight of a cubic foot of ice is 57½ pounds. Hence, if a block of ice were submerged below the surface of a stream it would, unless held down by some force, immediately rise to the surface. It is, however, true that ice frequently forms on the bottom and sides of canals, streams, and large rivers and remains submerged until a rise in temperature withdraws it from its moorings and causes it to float. In the United States this form of ice is commonly known as anchor ice or ground ice; in Canada it is called frazil ice; while geologists sometimes apply to it the term specular ice. According to Mr. Peterson, chief engineer of the Atlantic and Northwest Railway Company,[1] when soundings were made during the winter of 1881–82 across the channel of St. Lawrence River above Lachine Rapids and below Lake St. Louis

[1] See discussion of frazil ice in Trans. Can. Soc. Civ. Engrs., Vol. I, Pt. I, p. 20.

for the site of the present St. Lawrence Bridge, anchor ice from 2 to 3 feet deep was found adhering to the bed of the channel. At this bridge the river varies in depth from 5 to 40 feet. The bridge proper is nearly three-fourths of a mile long, and the current at low water varies from 2½ to 6 miles an hour. The anchor ice formed during a period of extremely cold weather, when the temperature was below zero. A subsequent rise in temperature detached it from the bed, when it immediately rose to the surface and floated downstream in the form of slush ice.

From observations in different parts of this continent, the conditions most favorable to the formation of anchor ice seem to be a water surface uncovered by surface ice, a temperature at or below zero, a clear sky, and moving water, caused either by wind on lake surfaces or by currents in stream channels. This form of ice is seldom found in the bed of a channel of water which is frozen over. It rises to the surface on cloudy days, and is usually produced, other conditions being favorable, during starry nights or clear days. It seldom forms in still water, but it is frequently found below rapids or fast-moving currents. A low temperature is the chief cause of this phenomenon. In still water in winter the bed of the channel of a stream or canal is usually several degrees warmer than the surface of the water. When, however, the bed is rough and the grade steep, the upper layers of water move much faster than those nearer the bottom, and instead of moving in straight lines parallel to the axis of the stream, the water advances in irregular curves. A particle of water would thus describe a path similar to a point on the surface of a revolving object of the desired form. At the time of the Johnstown flood, credible witnesses testified that they saw locomotives floating on the surface of the water. As a matter of fact, they could not float, but that they were brought to the surface for an instant by the force of the revolving current was well established. If we accept the statement that the motion of rapidly moving water over a rough surface is extremely complex, it follows that a particle of water which has become chilled by the cold at the surface will in a brief space of time be in contact with the bottom. In this way the temperature of the bed might be reduced considerably below the freezing point, and crystals of ice would then adhere to it in the same manner as to the extension rod of a reservoir valve, which in cold weather is often covered with a mass of ice crystals to the depth of an inch or more. When the wind blows cold over a lake surface, the upper 2 or 3 feet may become as cold as, if not colder than, the surface sheet of still water when ice begins to form. In the former case, the water is full of ice needles to a considerable depth; in the latter case, the needles are congregated in a horizontal plane at the surface.

To successfully operate canals in winter, attention should be given

to a few points which, as the writer has learned by experience, it is wise to observe. One of these somewhat general rules is to increase the flow prior to the beginning of the ice-forming period. Many water masters consider it safer to turn out most of the water before a cold spell, in the belief that the canal will be less likely to become choked and overflow. This is a mistaken course, for it exposes a larger proportion of the channel of the canal to the action of frost, and, other conditions being equal, a small flow will be converted into ice more rapidly than a large flow. It is better to maintain an average flow before the beginning of freezing weather, and if all of the water is not needed it may be allowed to flow through wasteways near the lower end. In this way the channel is kept free from frost, and if the head is not allowed to vary a thick coating of ice will in time form across the top. With such a covering and with no frost in the bottom of the canal, the writer has never known anchor ice to form.

It is true that slush ice may be admitted into the canal through the intake; but if care is exercised in by-passing or screening the greater part through wasteways at secondary gates, the remainder which may be allowed to enter the canal will not prove troublesome, since the temperature of the water beneath the ice will be, as a rule, sufficiently high to melt the ice crystals.

AQUATIC PLANTS IN CANALS.

Two years ago, at a farmers' institute, the following question was asked: Can the growth of water plants in irrigation canals be prevented, and, if not, how can such growth be most economically removed? In the discussion which followed various crude devices for the removal of moss, as it is termed, were described, and the fact that it grew most abundantly in canals of low grade and sluggish velocity was clearly brought out.

In all of the warmer States and Territories of the West the growth of vegetation in canals is the cause of much annoyance and expense to the irrigator. This is particularly true in New Mexico, Arizona, and California, and in the warmer portions of Colorado and Utah. In Montana, Wyoming, and Idaho the temperature of the irrigating waters is frequently too low to promote the growth of these plants. The disadvantages due to this cause consist in the loss of water, in additional trouble in subdividing the water among the several users, and in the greater risk of breaks. Unfortunately the moss grows most rapidly at the time when water is most needed to irrigate crops, and to be compelled to turn out the water, as many are at such a time, until the sun destroys the vegetation frequently results in a scarcity of water.

When no measures are taken either to remove or destroy this vegetation the channel becomes more or less choked, and the capacity may

be reduced in extreme cases to one-fifth the normal. It is also difficult under such conditions to divide the water equitably, for the reason that the water level on rating flumes and other measuring devices often rises as the quantity of water diminishes. In other words, the cross section of the water in the canal remains undiminished, or it may be increased, while the velocity is materially lessened. The fact that the water in the canals and flumes stands at a high mark while the volume flowing through is really below the normal is the cause of frequent breaks, since an attempt is usually made to crowd them beyond their safe capacity.

Of the scores of varieties of plants which grow in the water of canals it would be impossible to state with any degree of accuracy the natural conditions best adapted to the growth of each. Some varieties, for instance, seem to flourish in the coldest spring water, while others require a warm temperature. This doubtless accounts, in part at least, for the wide difference of opinion of superintendents of waterworks and irrigation canals in endeavoring to explain the presence of such vegetation in either the storage reservoir or the main canal. Speaking generally, one may say that the conditions most favorable to the growth of fresh-water plants are: (1) Clear water; (2) slow velocity; (3) a surface temperature above 60° F.; and (4) sunshine.

The writer can not now recall a single case in which so-called moss was found growing in turbid water. In 1886 and 1887, before the West Denver reservoir was roofed, immense quantities of fresh-water algæ formed in the clear water of the reservoir, and yet none was ever found in the canal and forebay 100 feet distant. South Platte River supplied both, but in the case of the reservoir the water was filtered through natural underground galleries.

Regarding the second general rule, numerous instances might be cited to show that moss grows more luxuriantly in canals of low grade, but two examples will suffice. The Logan, Hyde Park and Thatcher canal, diverting water from Logan River in Cache County, Utah, has a capacity of about 50 second-feet. Throughout the length of the main canal the grade is quite steep, with the exception of a distance of about 1,000 feet, where it is quite flat. Little, if any, moss grows on the steep portion, where the velocity is high; but on the flat portion the channel is in midsummer nearly filled with vegetable growth. The Bear River canal, in Boxelder County, Utah, branches, at a point some 6 miles below the dam, into what is known as the West canal and the Corinne canal. The West canal has a uniform grade of 1.056 feet to the mile for about 12 miles, and a bottom width at the upper end of 30 feet, which is reduced to 18 feet at the lower end. The Corinne canal has a grade of 2.112 feet to the mile throughout the upper portion, and a bottom width of 22 feet. On August 17, 1897, the West canal was carrying 87 second-feet of water, with a mean velocity of 1.48 feet per second. On August 21, 1897, the

TUNNEL OF BEAR RIVER CANAL, UTAH.

Corinne canal was carrying 110 second-feet, with a mean velocity of 2.36 feet per second. The effect of this difference in mean velocity is shown in the growth of the water plants. For years the West canal has been filled with vegetation from July 15 to September 1, necessitating frequent dredging, at considerable expense, while in the Corinne canal, which has a steeper grade, although moss grows to some extent, it has never demanded any special attention.

As has been stated, some species of these plants exist in cold water, but by far the greater number require water of a temperate heat in order to grow. This fact may readily be proved by noting the temperature of the surface water in reservoirs when moss first begins to appear. The conduit which supplies the distributive reservoir at Ogden, Utah, taps three different sources, viz, Cold Creek, Wheeler Creek, and Ogden River. On July 30, 1897, the temperature of the water in Cold Creek was 51° F., that of Wheeler Creek 55° F., and that of Ogden River 64° F., while the water at the surface of the reservoir was 59° F., with green algæ just beginning to cover portions of the water slope. On that day nearly all of the water in the reservoir came from the two creeks, and showed an increase in temperature of about 6°. If all of the water had been conveyed from Ogden River instead, with a corresponding increase in temperature, the water in the reservoir would have been 70°, and with water at so high a temperature the reservoir would soon have been filled with aquatic plants. Owing, however, to the mingling of cold water from snow-fed creeks with the warmer river water, the moss has never accumulated in sufficient quantities to necessitate removal except by skimming the surface, nor has it proved a nuisance, although it is now eight years since the reservoir was first filled and used. The absence of moss in most of the canals of Montana and Wyoming is doubtless due to the fact that the temperature of the water passing through the head gates remains low throughout the short irrigation period. In the evaporation tank at Bozeman, Montana, the writer noticed last summer that moss began to grow when the water in the tank reached a temperature of about 65° F.

Sunshine also is a prime requisite for plant growth. Water plants die when the rays of the sun are excluded. This was fairly well demonstrated in the summer of 1888, when the writer was building a roof over Cemetery Hill reservoir, at Denver, Colorado, in accordance with plans prepared by Mr. Charles P. Allen, chief engineer of the Denver Union Water Company. During hot weather in the month of June, when a space 44 feet along the center of the reservoir remained uncovered, moss began to grow in 14 feet of water under the exposed surface. When the roof was completed and the sunlight excluded all organic growth ceased to exist, and none has appeared since.

In locating irrigation canals consideration should be given to the grade, in order that the velocity may be sufficient to overcome the injuri-

ous effects of silt and vegetation. To accomplish this, the mean velocity should be not less than 2½ feet per second. When the mean velocity is less than 3 feet per second erosion of the canal bed seldom occurs, and in ordinary materials the engineer should fix the grade so as to produce a mean velocity of about 2¾ feet per second. When the physical features of the route are such that the desired velocity can not be obtained, the new canal should be built in a way to facilitate the removal of vegetable growth, by preventing any obstruction in the canal, such as fences, bridge posts, or trestles, and by removing sagebrush or willows from the banks in order to permit the free passage of teams.

Regarding canals already constructed and in operation, the first thing to be done is to ascertain the easiest method of removing the vegetation. In the smaller canals and ditches irrigators often take the old-fashioned scythe and snath and mow the weeds while the canal is full of

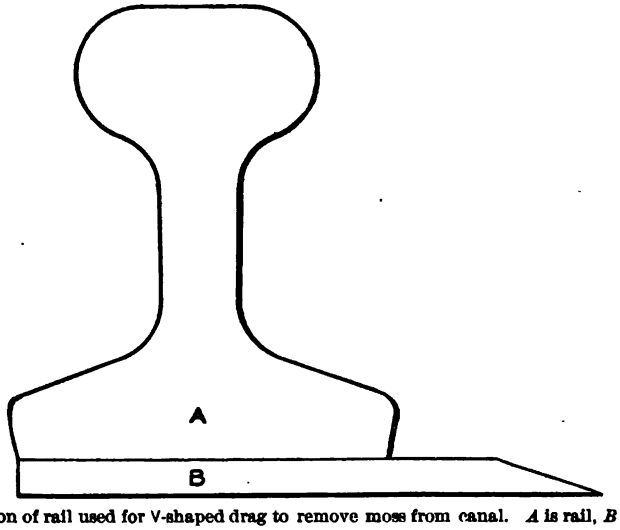


FIG. 4.—Section of rail used for V-shaped drag to remove moss from canal. *A* is rail, *B* is steel plate.

water. In other sections the proprietors prefer to turn out the water for a week or ten days and allow the sun to destroy the water plants. The latter plan is not to be recommended, on account of the serious injury done to the crops through the lack of water at a time when it is most needed; besides, many of the older canals have not a uniform bottom grade, and although the head gates may be closed stagnant water will usually be found in the channels over the greater part of the line. Harrows and disks have also been tried with some measure of success, but the chief objection to the use of such implements is that they break down and mix the weeds with the clay of the bed and sides instead of removing them.

At the time the writer was superintendent of the Bear River canal system his assistants, M. Mortensen and J. L. Rhead, designed a V-shaped drag (fig. 4) that was very effective. It was formed by bend-

ing a long 40-pound railroad rail into the shape of an inverted V, the base being extended to equal the bottom width of the canal. To the flanges of this rail there was riveted a plate of plow steel sharpened to a cutting edge, after the fashion of the grain-cycle section. At the apex of the V rail a chain was attached, to which were fastened two rope cables, which extended to either bank and to which teams were hitched. This contrivance was dragged up and down the canal until the plants were all cut and floated down the channel to the most convenient place of removal. In canals carrying more than 4 feet of water the apex of the drag and the rope cables should be fastened to a flat boat. In more shallow canals the apex should be fastened to the axle of a cart or wagon. This drag was used throughout the summer of 1898.

FLOW OF WATER.

During the summer of 1897 the writer was enabled, with the help of his assistants, T. H. Humphreys, A. P. Stover, and W. D. Beers, to make a number of experiments on the carrying capacities of irrigation ditches and canals. The funds necessary to carry on these investigations were provided by the United States Geological Survey and the agricultural experiment station of Utah. Shortly after the field work was completed, the writer resigned his position with the college to accept that of chief engineer and superintendent of the Ogden waterworks and the Bear River canal system, and since then his time has been so fully occupied with other duties that until now he has not had an opportunity to put into shape for publication the data collected two and a half years ago.

About sixty experiments were made on irrigation channels varying in size from the small ditch carrying a few miners' inches of water to the large canal carrying 225 second-feet. Experiments were made on nearly every form of ditch common to Western America, including many of the crudely formed ditches of the Mormon pioneers made nearly forty years ago, as well as the more modern and better designed canals of the Bear River canal system. The object sought was to ascertain as accurately as possible the present condition of ditches and canals that had been in operation a number of years. In order to determine the volume which flowed in any particular ditch and compare it with some well-known empirical formula, such as Kutter's or Chezy's, it was necessary to ascertain the slope of the surface of the water, the sectional area, the mean velocity, and the ratio between the water area and the wetted perimeter. This additional information regarding the form which channels assume after they are acted on by water and the atmosphere is valuable in that it gives the builder of a new canal some idea as to the proper form to adopt.

In the discussions which follow, the hydraulic elements, whether obtained in the field or computed from data taken in the field, have been referred to two well-known formulæ, Chezy's and Kutter's.

The carrying capacities of the conduits and irrigation canals of the West have been designed in accordance with Kutter's formula. It is therefore proper that experiments on the flow of water in canals should be compared with that formula.

Kutter did not live long enough to adapt his formula to American practice, particularly to American irrigation practice. His values of the coefficient of roughness (n) were confined to six different classes of channels, as follows:

Class I, n=0.010 for carefully planed boards or smooth cement.
Class II, n=0.012 for common boards.
Class III, n=0.013 for ashlar masonry.
Class IV, n=0.017 for rubble masonry.
Class V, n=0.025 for channels in earth, brooks, and rivers.
Class VI, n=0.030 for streams with detritus and aquatic plants.

It will be noted that only one of these six applies to irrigation canals. Besides, the mode of building and the character of the materials were different from those which prevail in America, and these differences would no doubt exert some influence on the results. However, the late P. J. Flynn, M. Am. Soc. C. E., of Los Angeles, California, took up the unfinished work of the noted Swiss engineer and adapted the experiments of Ganguillet and Kutter, as well as other hydraulicians, to American practice. Mr. Flynn's tables on the flow of water in open and closed channels have materially lessened the labors of every American hydraulic engineer, and his values for the coefficient of roughness (n) which we reproduce have been considered safe guides during the last ten years:

Table giving the value of coefficient of roughness (n) for different channels.[1]

n=.009, well-planed timber, in perfect order and alignment; otherwise, perhaps, .01 would be suitable.

n=.010, plaster in pure cement; planed timber; glazed, coated, or enamelled stoneware and iron pipes; glazed surfaces of every sort in perfect order.

n=.011, plaster in cement, with one-third sand in good condition; also for iron, cement, and terra-cotta pipes, well joined and in best order.

n=.012, unplaned timber, when perfectly continuous on the inside; flumes.

n=.013, ashlar and well-laid brickwork; ordinary metal; earthenware and stoneware pipe in good condition, but not new; cement and terra-cotta pipe not well joined nor in perfect order; plaster and planed wood in imperfect or inferior condition; and, generally, the materials mentioned with n=.010, when in imperfect or inferior condition.

n=.015, second-class or rough-faced brickwork; well-dressed stonework; foul and slightly tuberculated iron; cement and terra-cotta pipes, with imperfect joints and in bad order; and canvas lining on wooden frames.

n=.017, brickwork, ashlar, and stoneware in an inferior condition; tuberculated iron pipes; rubble in cement or plaster in good order; fine grave., well rammed, ½ to ¾ inches in diameter; and, generally, the materials mentioned with n=.013 when in bad order and condition.

[1] Flow of Water in Irrigation Canals, etc., by P. J. Flynn, C. E., pp. 19-20.

SANTA ANA CANAL (CEMENT-LINED), CALIFORNIA; CAPACITY 240 SECOND-FEET.

n=.020, rubble in cement in an inferior condition; coarse rubble, rough-set in normal condition; coarse rubble set dry; ruined brickwork and masonry; coarse gravel, well rammed, from 1 to 1½ inch diameter; canals with beds and banks of very firm, regular gravel, carefully trimmed and rammed in defective places; rough rubble, with bed partially covered with silt and mud; rectangular wooden troughs, with battens on the inside two inches apart; trimmed earth in perfect order.

n=.0225, canals in earth above the average in order and regimen.

n=.025, canals and rivers in earth of tolerably uniform cross-section, slope, and direction, in moderately good order and regimen, and free from stones and weeds.

n=.0275, canals and rivers in earth below the average in order and regimen.

n=.030, canals and rivers in earth in rather bad order and regimen, having stones and weeds occasionally, and obstructed by detritus.

n=.035, suitable for rivers and canals with earthen beds in bad order and regimen, and having stones and weeds in great quantities.

n=.05, torrents encumbered with detritus.

One of the objects which the writer had in view in making his experiments was to compare the results, particularly the values of the coefficient of roughness (n), with those given by Mr. Flynn for canals in similar condition. The results are herein given (pages 27 to 45) in eight groups, beginning with canals which were in excellent condition and ending with those which were in very poor condition.

The field work consisted in selecting a suitable portion of the canal to be investigated, in writing a brief description of the prevailing conditions, and in ascertaining the discharge in second-feet, the slope of the water surface, the sectional area, and the wetted perimeter. The office work consisted in computing, from the data taken in the field, the mean velocity, the hydraulic mean radius, the coefficient of roughness (n) as given in Kutter's formula, and the general coefficient (c) as given in Chezy's formula where $V = C\sqrt{RS}$.

The discharge was measured either by a current meter or by a trapezoidal weir. The trapezoidal weir designed by the Italian engineer, Cesare Cippoletti, was preferred to the Francis rectangular weir, on account of the simplicity of calculation as well as the probable greater accuracy. The one general equation used in all weir calculations was $Q=3.367\ LH^{\frac{3}{2}}$, where Q equalled the discharge in second-feet, L the bottom length of the weir in feet, and H the depth of the water in feet over the crest of the weir. The conditions necessary for accurate measurement were carefully observed, such as a low velocity of approach, a free fall, complete contraction on the bottom and sides, and a close measurement of the head of water over the weir.

In determining the discharge by means of the current meter the results were not so accurate, but in every case from three to six separate and distinct current-meter measurements were made, and it was thought that the mean of all of these measurements would not vary much from the actual discharge. At the close of the field work the

current meter was carefully rerated at the rating station in Washington, and the new rating table was used in the calculation of the velocities of flow in the canals tested.

The average cross section was obtained by plotting, in different colors and on a large scale, the three or more cross sections taken in the field. A new perimeter was then adopted from the typical cross section, which represented the average of the cross sections plotted. Its length was found by a pair of dividers, its area by a planimeter. The depths of water given in the tables are taken from the typical cross section, but approach very nearly to an average of all of those taken in the field at any one point.

The slope of a canal, represented by the fall of a given portion, usually from 100 to 200 feet, divided by the distance, was determined by a new Buff and Berger 18-inch level and a leveling rod reading to thousandths of a foot. The slope of a water surface is difficult to ascertain with accuracy, for the reason that in nearly all channels there are pulsations which cause the surface to rise and fall. Again, in irrigation canals long in use the bottom grade, owing either to abrasion or sedimentation, is seldom uniform, and the flow of water in any comparatively short portion is more or less influenced by the velocity in the section above; e. g., if a portion 100 feet in length had a fall of 4 feet to the mile, and another portion just below had a fall of 3 feet to the mile, the influence of the steeper grade of the higher portion would be felt on the lower portion. One could partially eliminate the error from this source by taking a long distance as a test, but this would introduce greater errors due to alignment and diversity of cross section. The method followed in determining the slope was to drive small wire finishing nails into the tops of submerged oak stakes at each end of the section to be tested. It was not always possible to have the top of the nail coincide exactly with the surface of the water, but this introduced no error in the results, provided the heads of both nails occupied the same relative position to the surface of the water. In the case of pulsations or slight waves caused by winds, the tops of both nails were even with either the highest or the lowest water surface. Under ordinary conditions this method will give as accurate results as those obtained by hook gages.

Having obtained all of the hydraulic elements, either from data taken in the field or from calculations made in the office, the coefficients c in the Chezy formula and n in the Kutter formula were then computed and checked.

The following are the results of 42 of 60 experiments made, arranged, as already stated, in eight groups, beginning with canals which were in excellent condition and ending with those which were in very poor condition.

CEMENT-LINED DISTRIBUTING DITCH AT NORTH POMONA, LOS ANGELES COUNTY, CALIFORNIA

GROUP No. 1.

[n equals 0.0134 to 0.0184.]

Experiment No. 21.—This experiment was made on the main line of the Bear River canal, about 7 miles below the dam and about 200 feet above the division gates. Water was first turned into this portion of the system by the writer in the spring of 1891, and during the six years of operation the action of the water had changed the section from a trapezoidal form having a bottom width of about 15 feet and side slopes of 1 to 1 to that of the segment of an ellipse, as shown in fig. 5. The formation is clayey loam and the channel when measured was covered with a coating of sediment but was entirely free from vegetation, gravel, or pebbles, and was of regular cross section and in excellent condition. Three current-meter measurements were made, with the following results:

	Second-feet.
First measurement	226.56
Second measurement	224.94
Third measurement	225.14
Average	225.55

The results of this experiment are given in the table on page 30.

Experiment No. 12.—This experiment was made on the Bear River City canal, 900 feet below the head gates. This canal is a branch of the Bear River canal system, and the conditions regarding materials, length of time in operation, and coating of fine sediment were similar to those of experiment No. 21. When first made, this channel was trapezoidal, with 1 to 1 slopes, but it has since changed to the form outlined in fig 5. Current-meter measurements were made at the top, middle, and bottom of the portion tested, with the following results:

	Second-feet.
Top measurement	10.54
Middle measurement	10.91
Bottom measurement	10.58
Average	10.68

For the results of this test see the table on page 30.

Experiment No. 38.—This test was made on a lateral of the Bear River canal in that part of the valley known as Roweville, Utah. This lateral was in good condition and was lined with clayey sediment similar to that referred to in the previous descriptions. The discharge was obtained by means of a trapezoidal weir, the bottom length of which was 3.15 feet. The depth of water on the crest was 0.822 foot. The cross section is shown in fig. 5. For the results of the test see the table on page 30.

Experiment No. 15.—The place selected for this experiment was on the Corinne branch of the Bear River canal, about 4 miles below the division gates. This canal had been operated six years, and although

its section when new was horizontal on the bottom, with side slopes in excavation of 1 to 1, its form at the time of the test approached the

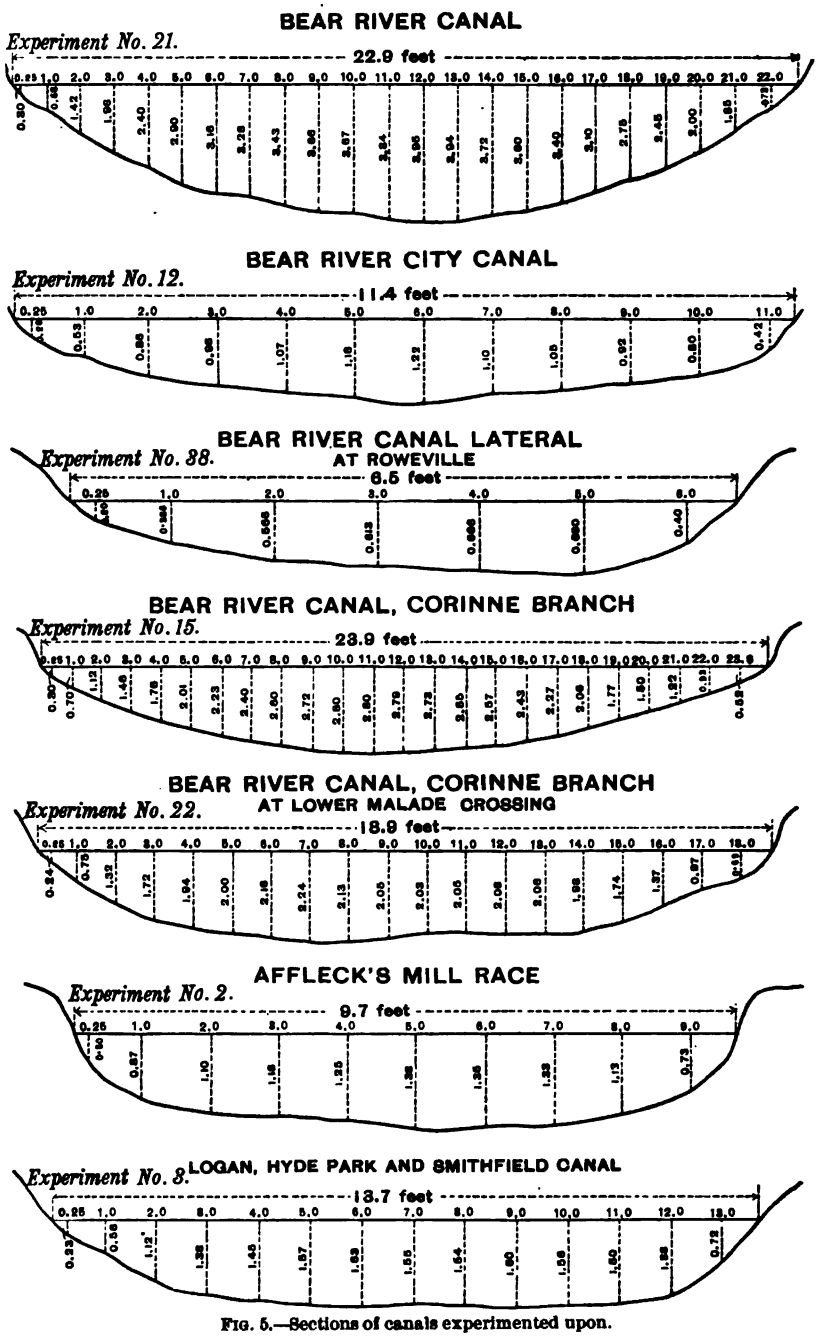


FIG. 5.—Sections of canals experimented upon.

segment of an ellipse, as may be seen by a reference to fig 5. Where the test was made the canal was entirely free from vegetation, and

there was nothing to obstruct the flow except the friction on the smooth perimeter, which was coated with fine silt. Meter measurements were made at the top, middle, and bottom of the portion chosen, which was 100 feet in length, with the following results:

	Second-feet.
Top measurement	109.09
Middle measurement	109.52
Bottom measurement	110.07
Average	109.56

For the results of the test see the table on page 30.

Experiment No. 22.—This test was made on the Corinne branch, some 6 miles below the place where the last test was made, at a point about 1,300 feet below the Lower Malade crossing. (Fig. 5.) The date of the experiment was September 9, 1897, which accounts for the small volume in the canal, the irrigation season in that section being nearly past. This canal is similar to those previously described, and was in excellent condition at the time of the experiment. The formation was a clayey loam, and the action of the water for six summers had left the channel quite smooth and well coated with silt. The discharge was obtained by meter measurement in the usual manner. The results of the test are given in the table on page 30.

Experiment No. 2.—This test was made on a 60-foot length of Affleck's mill race, near Logan, Utah. The general form of the cross section is shown in fig. 5. The channel was composed of gravel ranging in size from small particles to one-half inch in diameter, with an occasional pebble 2 inches in diameter. The sides were in fair condition, with some weeds near the edges which did not, however, interfere to any appreciable extent with the flow of water. The discharge was obtained from the mean of the following current-meter measurements:

	Second-feet.
Upper measurement	15.152
Do	15.360
Middle measurement	15.574
Do	15.870
Lower measurement	15.283
Do	14.836
Average	15.346

The results of the test are given in the table on page 30.

Experiment No. 3.—On June 23, 1897, a site for an experiment was selected on the Logan, Hyde Park and Smithfield canal (fig. 5), near the mouth of Logan Canyon, Cache County, Utah. Water was first turned into this canal in 1882, and at the time of the experiment it had been operated fifteen years. The bottom and sides were smooth and composed of earth and gravel. The particles of gravel varied in size from one-half inch to 1 inch in diameter, with some pebbles

canal (see fig. 6), built in 1864–1867, and the largest ditch diverting water from Logan River. The bottom and sides, which were smooth

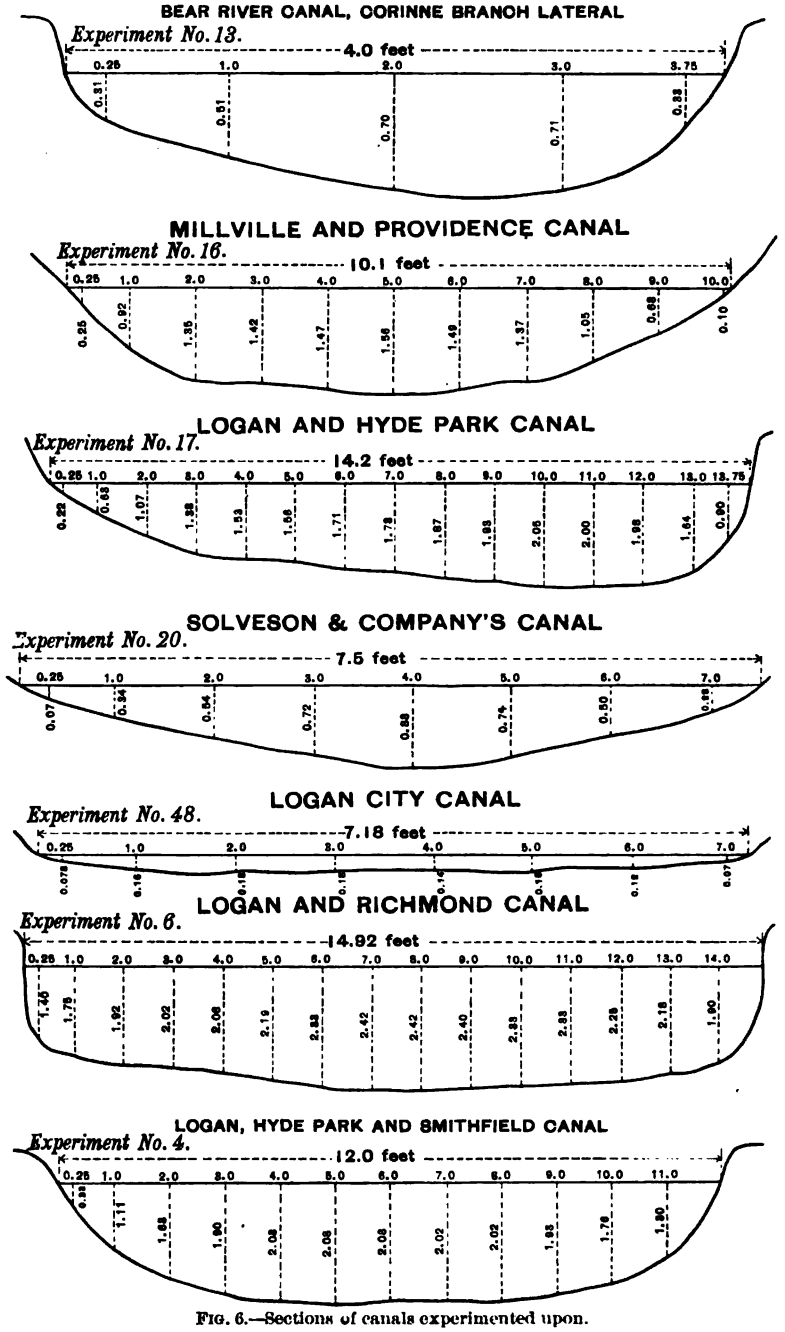


FIG. 6.—Sections of canals experimented upon.

and free from vegetation, were composed of a clayey loam. With the exception of some indentations on the sides near the top the canal was

in good condition. Meter measurements were made at three points in the 100-foot section, with the following results:

	Second-feet.
Top measurement	68.59
Do	68.76
Middle measurement	68.80
Do	68.75
Bottom measurement	67.90
Do	68.58
Average	68.56

For results see table on this page.

Experiment No. 4.—This test was made on a 60-foot section of the Logan, Hyde Park and Smithfield canal near its point of diversion in Logan Canyon (fig. 6). The channel was composed of well-packed, coarse gravel and small cobbles, the common sizes being 1 inch, 2 inches, and 3 inches in diameter, in about equal proportion. There were some weeds on one edge, but it is doubtful whether they retarded the flow. This canal had been operated since 1882. The following are the results of the current-meter measurements:

	Second-feet.
Top measurement	51.09
Do	51.01
Middle measurement	51.52
Do	51.07
Bottom measurement	51.92
Do	51.57
Average	51.36

The results of the test are given in the following table:

Table showing values of hydraulic elements in group No. 2.

Experiment.	Discharge.	Area of water section.	Mean velocity.	Hydraulic mean radius.	Wetted perimeter.	Coefficient of roughness (n).	Slope.	Coefficient (c).*
	Sec.-feet.	*Sq. feet.*	*Feet per second.*		*Lineal feet.*		*Feet per foot.*	
No. 13	2.53	2.23	1.14	0.50	4.46	0.0194	0.00068	61.55
No. 16	22.28	11.50	1.94	1.07	10.80	0.0195	0.00062	74.63
No. 17	23.55	21.78	1.08	1.40	15.60	0.0197	0.00015	75.72
No. 20	4.03	4.00	1.01	0.52	7.73	0.0201	0.00056	59.22
No. 48	0.56	1.03	0.54	0.14	7.19	0.0204	0.00135	39.29
No. 6	68.56	32.02	2.14	1.74	17.80	0.0211	0.00046	75.63
No. 4	51.36	20.61	2.49	1.52	13.56	0.0213	0.00077	73.05

*In Chezy's formula.

GROUP No. 3.

[n equals 0.0218 to 0.0238.]

Experiment No. 11.—This experiment was made on the Point Lookout canal (fig. 7) when the volume carried (87.29 second-feet) was small

compared with the maximum capacity, which is nearly 600 second-feet. The surface width was nearly 36 feet; the depth of water at the deepest place about 2¼ feet. The section chosen was entirely free from aquatic plants, but more or less vegetation was to be found both above and below this site. The channel was smooth and composed of a clayey loam lined with sediment. The current-meter measurements were as follows:

	Second-feet.
Top measurement	87.64
Middle measurement	87.82
Bottom measurement	86.40
Average	87.29

For results of test see table on page 37.

Experiment No. 29.—The Providence Town canal (fig. 7) was only part full when measured by weir on September 16, 1897. The length of the weir was 2 feet, the depth of water 0.376 foot. The bed was composed of well-packed and smooth gravel about the size of Spanish nuts, embedded in sand and sediment. There was no vegetation. The ditch had been in use about thirty years. The results of test are given in the table on page 37.

Experiment No. 19.—The Lewiston canal (fig. 7), begun in 1860 and completed in 1880, diverts water from Cub River and serves the bench lands near Franklin, Idaho, and Lewiston, Utah. It is capable of conveying about 125 second-feet when running full, but at the time of the experiment it contained only 32.72 second-feet. The channel consisted of smooth, light-colored clay, but about one-fifth of the perimeter was covered with a growth of fibrous moss, locally termed "frog moss." The meter measurements were as follows:

	Second-feet.
Top measurement	33.22
Do	32.92
Middle measurement	32.94
Do	32.54
Bottom measurement	32.33
Do	32.39
Average	32.72

For results see table on page 37.

Experiment No. 31.—This experiment was made on the Providence Upper canal (fig. 7), near the town of Providence, Utah. The conditions were similar to those of No. 29. The flow was measured over a weir 2 feet in length and having 0.494 foot of water flowing over the crest. The results of test are given in the table on page 37.

Experiment No. 14.—This test was made on a lateral of the Bear River canal near Central farm (fig. 7). The formation was a clayey loam; the water channel was coated with sediment, and also contained

patches of what is locally termed "horsetail moss"—an aquatic plant which grows to a length of 5 or more feet. The presence of this

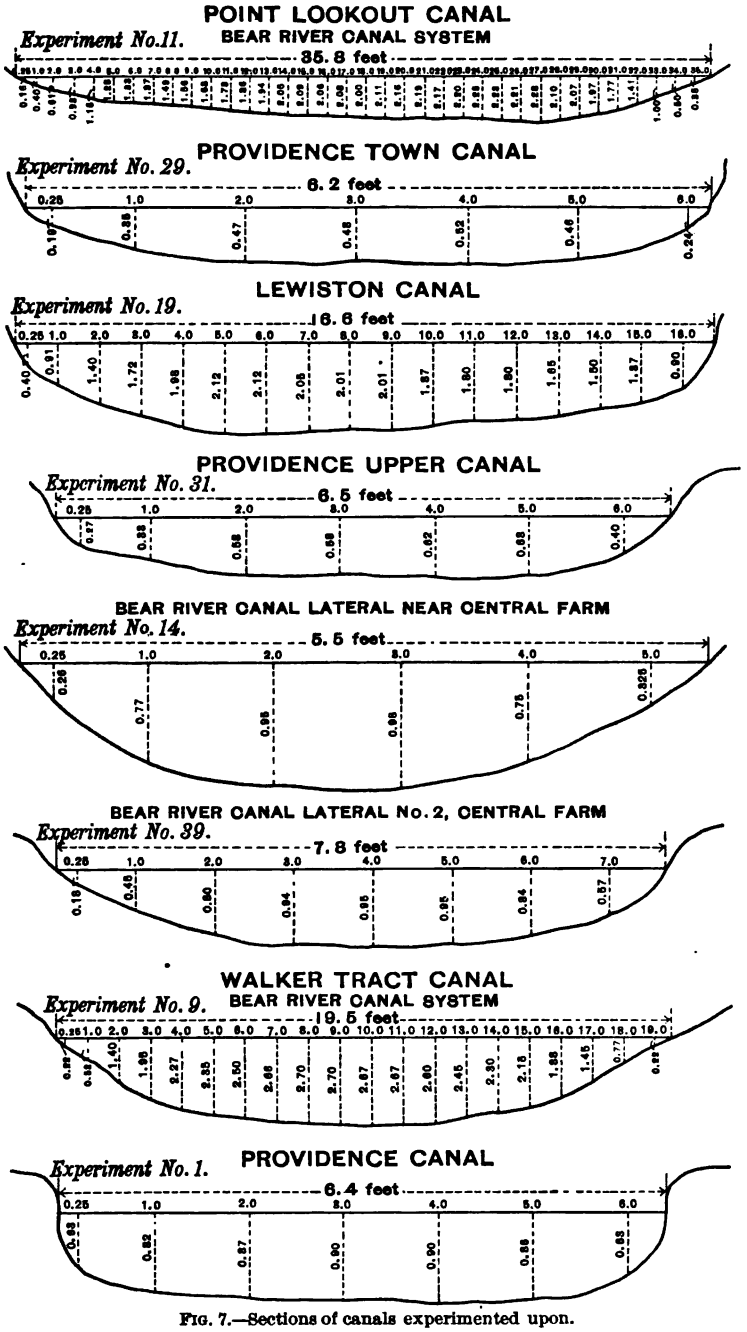


FIG. 7.—Sections of canals experimented upon.

plant, together with somewhat uneven edges, doubtless retarded the velocity.

The discharge was obtained from the average of three meter measurements, which were as follows:

	Second-feet.
Top measurement	4.69
Middle measurement	4.67
Bottom measurement	4.54
Average	4.63

For results see table on page 37.

Experiment No. 39.—In this instance a portion, 80 feet in length, of the Central farm lateral No. 2 (fig. 7) in the vicinity of the section tested in experiment No. 14 was used. The conditions were similar to No. 14, except that there was no moss in the bed; a few bunches of grass were, however, scattered along the edges. Clay and sediment formed the bottom. The discharge was measured over a weir. The results of the test are given in the table on page 37.

Experiment No. 9.—The Walker Tract canal (fig. 7), a continuation of the Point Lookout canal, was built in 1892, to carry a much larger volume of water than indicated in this experiment. The true grade was between 3 and 4 feet to the mile, but at the time of the experiment the irrigators had inserted dams in the channel at different points, to raise the water surface. This partly accounts for the low grade of 0.63 foot to the mile, although this figure may be too low, owing to the action of waves on the canal at the time the elevations were taken. The channel consisted of clayey loam, and it was free from vegetation. The meter measurements were as follows:

	Second-feet.
Top measurement	38.70
Middle measurement	38.72
Bottom measurement	38.23
Average	38.55

The results of the test are given in the table on page 37.

Experiment No. 1.—This test was made on the Providence canal (fig. 7), which diverts water from Logan River. The bed was composed of gravel about the size of peas, with other particles about the size of small walnuts scattered about. There was no vegetation, and the discharge was found by taking the average of the following meter measurements:

	Second-feet.
Top measurement	10.21
Do	9.95
Middle measurement	9.86
Do	10.20
Bottom measurement	9.75
Do	9.93
Average	9.98

The results of the test are given in the following table:

Table showing values of hydraulic elements in group No. 3.

Experiment.	Discharge.	Area of water section.	Mean velocity.	Hydraulic mean radius.	Wetted perimeter.	Coefficient of roughness (n).	Slope.	Coefficient(c).*
	Sec.-feet.	*Sq. feet.*	*Feet per second.*		*Lineal feet.*		*Feet per foot.*	
No. 11..	87.29	58.94	1.48	1.60	36.73	0.0218	0.00027	71.19
No. 29..	1.55	2.55	0.61	0.40	6.34	0.0223	0.00040	48.00
No. 19..	32.72	27.53	1.19	1.52	18.08	0.0224	0.00020	68.07
No. 31..	2.34	3.29	0.71	0.48	6.80	0.0229	0.00043	49.34
No. 14..	4.63	3.89	1.19	0.65	6.00	0.0230	0.00075	54.03
No. 39..	7.90	5.74	1.38	0.71	8.12	0.0230	0.000875	55.38
No. 9..	38.55	38.32	1.01	1.83	20.90	0.0232	0.00012	67.83
No. 1..	9.98	5.26	1.90	0.71	7.40	0.0238	0.00175	53.85

* In Chezy's formula.

GROUP No. 4.

[n equals 0.0238 to 0.0260.]

Experiment No. 5.—This test was of a 50-foot length of the College and City canal (fig. 8), a branch of the Logan, Hyde Park and Smithfield canal. The bed was of coarse gravel, ranging in size from particles one-half inch to 2 inches in diameter, more or less, embedded in finer materials. The edges were somewhat irregular, with some willow roots, but there was no vegetation in the channel. The meter measurements were as follows:

	Second-feet.
First trial	7.73
Second trial	7.54
Third trial	7.48
Fourth trial	7.63
Average	7.60

The results are given in the table on page 39.

Experiment No. 27.—This test was of a 100-foot length of the Logan, Hyde Park and Thatcher canal (fig. 8), near Logan, Utah. The sides of the channel were smooth and coated with sediment. The bottom consisted of earth, gravel, and pebbles, some of the latter being 2½ inches in diameter. The coarser material covered about one-fourth of the perimeter. For results see table on page 39.

Experiment No. 18.—This experiment was also made on the College and City canal (fig. 8). There was no vegetation to check the velocity, but the sides were uneven, and the bed was covered with fragments of disintegrated flat rock, ranging in size from one-half inch to 2 inches across the greatest dimension. The meter measurements were as follows:

	Second-feet.
Top measurement	5.69
Middle measurement	5.67
Bottom measurement	5.61
Average	5.66

The results are given in the table on page 39.

Experiment No. 63.—The discharge of the small ditch known as the Hyrum canal (fig. 8), on which this experiment was made, was meas-

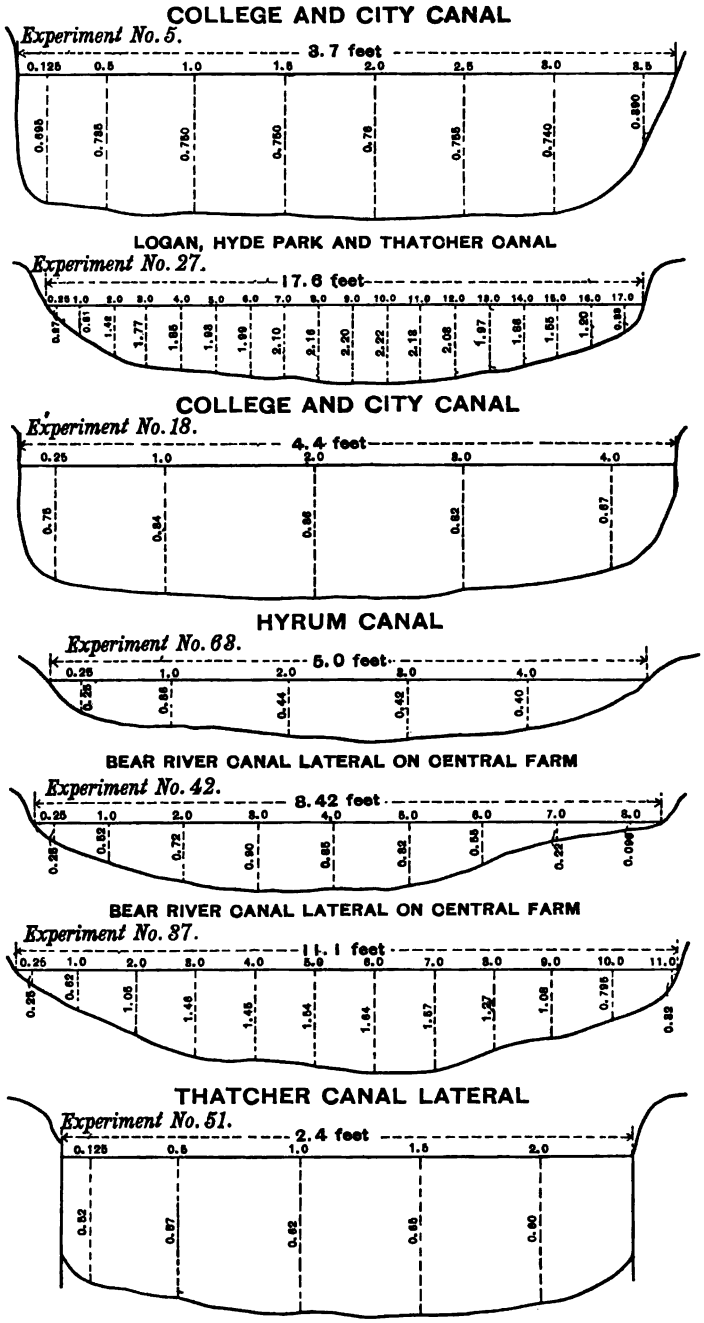


FIG. 8.—Sections of canals experimented upon.

ured over a weir. The sides of the channel were of earth, but about half of the perimeter along the bottom was covered with rock frag-

ments from ½ inch to 1 inch across. Weeds and alfalfa grew up to the edge of the water. The results are given in the following table:

Table showing values of hydraulic elements in group No. 4.

Experiment.	Discharge.	Area of water section.	Mean velocity.	Hydraulic mean radius.	Wetted perimeter.	Coefficient of roughness (*n*).	Slope.	Coefficient (*c*).*
	Sec.-feet.	*Sq. feet.*	*Feet per second.*		*Lineal feet.*		*Feet per foot.*	
No. 5..	7.60	2.59	2.93	0.55	4.73	0.0238	0.00616	50.43
No. 27..	60.23	30.60	1.97	1.62	18.90	0.0246	0.0006	63.16
No. 18..	5.66	3.51	1.61	0.65	5.44	0.0247	0.0016	50.18
No. 63..	1.57	1.87	0.84	0.35	5.28	0.0260	0.0013	38.92

* In Chezy's formula.

GROUP No. 5.

[*n* equals 0.0293 to 0.0319.]

Experiment No. 42.—This experiment was made on a Central farm lateral of the Bear River canal (fig. 8). The discharge was measured over a weir. The bed was composed of clay, but the cross section was not uniform, and there were a few bunches of grass scattered along the edges.

Experiment No. 37.—This experiment also was made on one of the Central farm laterals (fig. 8), and nearly the same conditions were found as existed in the preceding experiment.

Experiment No. 51.—This experiment was on a small lateral of the Thatcher canal (fig. 8). There was a narrow board on each edge, as shown in the figure. The bottom was coarse gravel and cobbles, ranging in size from large peas to goose eggs. The discharge was determined by weir measurement.

Experiment No. 64.—This experiment was on the Hyrum canal (fig. 9), which at the time of the test carried only a large irrigation stream. The flow was measured over a weir. The sides were considerably overgrown with alfalfa and weeds, and the bed consisted of cobbles from the size of hazelnuts to that of walnuts.

Table showing values of hydraulic elements in group No. 5.

Experiment.	Discharge.	Area of water section.	Mean velocity.	Hydraulic mean radius.	Wetted perimeter.	Coefficient of roughness (*n*).	Slope.	Coefficient (*c*).*
	Sec.-feet.	*Sq. feet.*	*Feet per second.*		*Lineal feet.*		*Feet per foot.*	
No. 42..	8.08	4.79	1.69	0.56	8.56	0.0293	0.0033	39.24
No. 37..	7.83	12.79	0.61	1.08	11.80	0.0295	0.00017	45.08
No. 51..	1.08	1.44	0.75	0.44	3.25	0.0310	0.00113	33.51
No. 64..	2.47	2.82	0.88	0.49	5.79	0.0319	0.0014	33.55

* In Chezy's formula.

GROUP NO. 6.

[n equals 0.0329 to 0.0365.]

Experiment No. 55.—This experiment was on a Smithfield canal lateral (fig. 9) in Cache County, Utah. The bed consisted of cobbles partially covered with silt. The edges had been made irregular by cattle. The discharge was measured over a weir.

Experiment No. 47.—This experiment was on a small lateral of the Logan and Hyde Park canal (fig. 9) on one of the streets of Smithfield, Cache County, Utah. It will be noted, by referring to the table on this page, that the fall is about 1 per cent, but that the mean velocity is only 1.35 feet per second. This small channel was composed wholly of loose, coarse gravel, varying in size from that of a pea to that of a small hen's egg. The discharge was measured over a weir.

Experiment No. 26.—This experiment was on the Logan and Benson-Ward canal (fig. 9), in Cache County, Utah. In this case the flow was much impeded by horsetail moss, which covered about three-fourths of the water section. The bed was composed of medium-sized gravel, and the discharge was ascertained by meter measurement.

Experiment No. 33.—This is another instance in which a small volume of water flowed very slowly over a rough surface, notwithstanding that the fall was 64 feet to the mile. This experiment was on a lateral of the Hyrum canal (fig. 9), near the town of Hyrum, Cache County, Utah. The channel consisted chiefly of rock, ranging in size from 1 inch to 3 inches in diameter. The flow was measured over a weir.

Table showing values of hydraulic elements in group No. 6.

Experiment.	Discharge.	Area of water section.	Mean velocity.	Hydraulic mean radius.	Wetted perimeter.	Coefficient of roughness (n).	Slope.	Coefficient (c).*
	Sec.-feet.	*Sq. feet.*	*Feet per second.*		*Lineal feet.*		*Feet per foot.*	
No. 55..	1.29	2.97	0.43	0.52	5.70	0.0329	0.000353	32.02
No. 47..	0.85	0.63	1.35	0.27	2.32	0.0337	0.00991	25.87
No. 26..	24.59	29.14	0.84	1.32	22.00	0.0352	0.00033	40.33
No. 33..	0.81	0.80	1.02	0.20	4.00	0.0365	0.01212	20.68

* In Chezy's formula.

GROUP NO. 7.

[n equals 0.0377 to 0.0424.]

Experiment No. 57.—This test was on a small lateral of the Smithfield City canal (fig. 9), which had a steep grade and a rough channel, made up principally of coarse gravel and cobbles. The flow was measured over a weir. (See table on page 42.)

Experiment No. 62.—At the time of the test this small lateral of the Hyrum canal (fig. 9) was partially overgrown with alfalfa, and there

was a strip of moss on each side which covered about one-fifth of the channel, which was made up of flat fragments of rock from 1 inch to

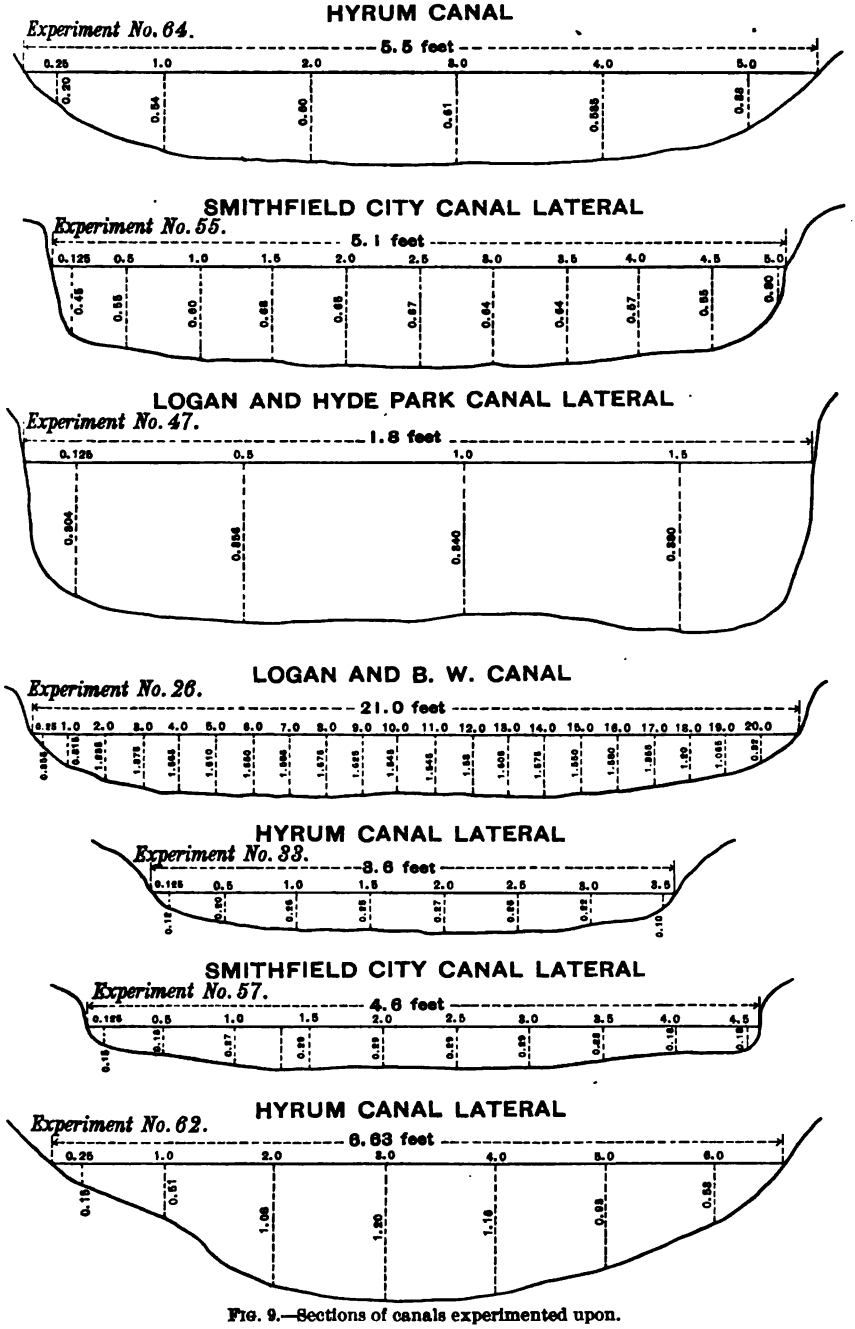


FIG. 9.—Sections of canals experimented upon.

3 inches across the longest diameter. A weir was used to determine the discharge.

Experiment No. 56.—This experiment was on a lateral of the Smithfield City canal (fig. 10) at Smithfield, Utah. The conditions were similar to those of No. 57. The edges of the lateral were rough and uneven, and the bed a mass of pebbles and cobbles washed clean by the large flow in the early part of the season. (See table below.)

Experiment No. 24.—This test was on a well-formed canal (fig. 10), the property of the Brigham City Electric Light Company, located near the mouth of Boxelder Canyon, in the county of the same name. The bed consisted of medium-sized gravel, unpacked, and about one-third of the water area was filled with long, waving, water plants, resembling horsetails. The presence of these plants retarded the velocity, and no doubt changed the degree of roughness from a value of about 0.020 to that given in the table—0.0424. Only two meter measurements were made, but these check very closely; they were 30.99 and 31.16 second-feet. The results are given in the following table:

Table showing values of hydraulic elements in group No. 7.

Experiment.	Discharge.	Area of water section.	Mean velocity.	Hydraulic mean radius.	Wetted perimeter.	Coefficient of roughness (n).	Slope.	Coefficient (c).*
	Sec.-feet.	*Sq. feet.*	*Feet per second.*		*Lineal feet.*		*Feet per foot.*	
No. 57..	1.48	1.11	1.33	0.23	4.75	0.0377	0.0171	21.07
No. 62..	2.47	5.55	0.44	0.77	7.20	0.0393	0.00029	29.62
No. 56..	1.17	1.06	1.10	0.23	4.70	0.0423	0.0170	17.93
No. 24..	31.07	20.44	1.52	1.62	12.60	0.0424	0.00115	35.19

* In Chezy's formula.

GROUP No. 8.

[n equals 0.0469 to 0.0529.]

The results of the experiments in this group are given in the table on page 44.

Experiment No. 35.—This experiment was made on a small stream (fig. 10) which supplies the Temple grounds in Logan, Utah. The natural channel was made up of sand and silt, but at the time the experiment was made it was more than one-third full of horsetail moss. The discharge was ascertained by weir measurement.

Experiment No. 46.—This experiment was on a canal at Brigham, Utah (fig. 10), which was more than half filled with horsetail moss. The edges were also overgrown with watercress and weeds, but the part of the bottom which was exposed was covered with fine gravel.

Experiment No. 53.—This test was on a small lateral of the Thatcher canal (fig. 10) near Logan, Utah. About two-thirds of the lateral was more or less covered and filled with horsetail moss. The remainder of the exposed surface of the bed was sediment.

Experiment No. 52.—This test was on another small lateral of the Thatcher canal (fig. 10). The bed was composed of fine silt, but about

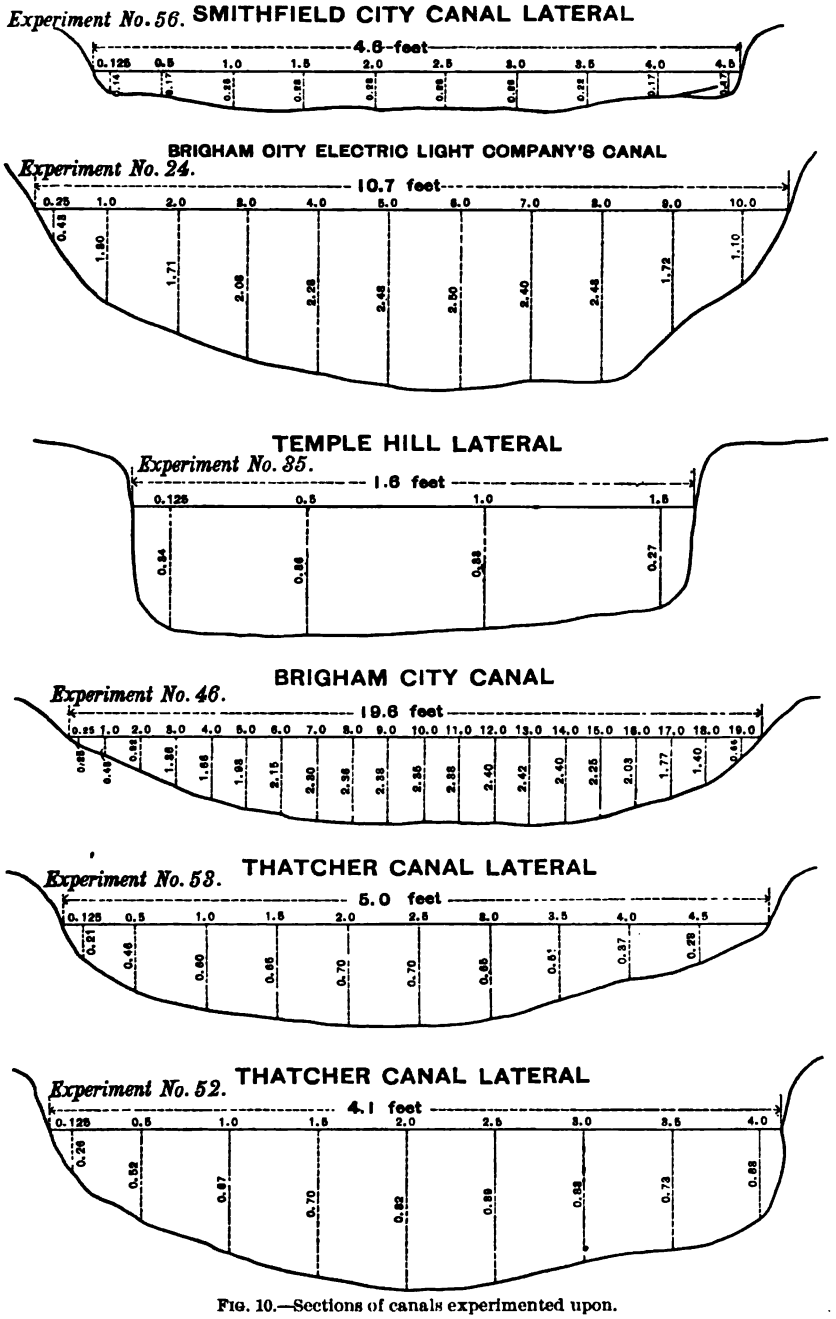


FIG. 10.—Sections of canals experimented upon.

three-fourths of the entire channel was covered with aquatic plants similar to those described in the preceding experiments.

Table showing values of hydraulic elements in group No. 8.

Experiment.	Discharge.	Area of water section.	Mean velocity.	Hydraulic mean radius.	Wetted perimeter.	Coefficient of roughness (n).	Slope.	Coefficient (c).*
	Sec.-feet.	*Sq. feet.*	*Feet per second.*		*Lineal feet.*		*Feet per foot.*	
No. 35..	0.28	0.52	0.54	0.25	2.10	0.0469	0.00442	16.28
No. 46..	23.40	35.76	0.65	1.74	20.50	0.0499	0.00028	29.60
No. 53..	1.08	2.57	0.42	0.48	5.30	0.0519	0.00107	18.41
No. 52..	1.08	2.90	0.37	0.58	4.98	0.0529	0.00064	19.16

*In Chezy's formula.

The writer presents these results of experiments in the hope that they may aid those building and operating irrigation systems in the West to arrive at a better understanding regarding the behavior and carrying capacities of irrigation canals. In planning new systems it is necessary to know the approximate volume of water which each new channel will carry. As has been stated, the formula most generally used for this purpose is that of Kutter. But Kutter's formula will give results either too great or too small, in proportion as one chooses too low or too high a value for the coefficient of roughness (n).

The experiments made in the past to determine the values of n for canals have been largely confined either to new channels or to conditions somewhat different from those which prevail in Western America. In the case of new canals it is believed that the coefficient of roughness of the wetted surface is much greater than it is in canals of similar form and materials when the wetted surface is well coated with fine sediment. The friction seems to depend quite as much on the way in which the gravel and cobbles are packed as on their size. Again, the effect of water plants in retarding the velocity of water in canals has usually been considered less than our experiments indicate.

On account of the dissimilarity between the physical conditions of the channels from which the present values of n have been derived and the ditches and canals of irrigated America, the writer has attempted, in the following table, to assign values for n which are more in accordance with the conditions now existing in the Rocky Mountain States. Future experiments, however, in which the details are more carefully looked after, may modify these values of n.

Values of n for irrigation ditches and canals.

n=0.0175 for canals in earth in excellent condition, well coated with sediment, regular in cross section, and free from vegetation, loose pebbles, and cobbles.

n=0.020 for canals in earth, in good condition, lined with well-packed gravel, partly covered with sediment, and free from vegetation.

n=0.0225 for canals in earth, in fair condition, the wetted surface being lined with sediment, with an occasional patch of minute algæ, or composed of loose gravel without vegetation.

n=0.0250 for canals in earth, in average condition, having few sharp bends, and being fairly uniform in cross section; the water slopes and bottom being lined with sediment and minute algæ, or composed of loose, coarse gravel and fragments of rock less than 2 inches in diameter, and free from vegetation.

n=0.0275 for canals in earth, below the average in grade, alignment, and cross section, having indentations on the sides, the edges in places being partially filled with earth and gravel, and the lining composed of coarse gravel and cobbles unpacked. This value would also apply to a smooth, regular surface if the channel were partially filled with aquatic plants.

n=0.0300 for canals in earth, in rather poor condition, having the bed partially covered with débris, or having comparatively smooth sides and bottom, with bunches of grass and weeds projecting into the water and with aquatic plants growing in the channel.

n=0.0350 for small ditches having a rough, uneven bed, and for canals in earth in fairly good condition, but partially filled with aquatic plants.

n=0.040 for canals in earth the channels of which are about half full of aquatic vegetation.

n=0.050 for canals in earth the channels of which are about two-thirds full of aquatic vegetation.

The writer's experiments on the flow of water in irrigation ditches and canals, herein briefly described, seem to justify the following conclusions:

(1) That sections of canals in earth, although carefully built, of a trapezoidal form, with the bottom width horizontal, soon change to segments resembling those of an ellipse.

(2) That in all large or medium-sized canals in earth berms are necessary in order to prevent a portion of the excavated material from rolling into the canal.

(3) That during the first season of their operation the carrying capacities of irrigation canals are less than during subsequent seasons, provided the same conditions are maintained.

(4) That the coefficient of friction in canals well lined with sediment, in good condition and long in use, is less than usually has been supposed.

(5) That the frictional resistance of coarse materials, such as gravel, pebbles, or cobbles, depends to a large extent on whether such material is well packed or loose.

(6) That roughness in a small ditch exerts a greater influence in retarding flow than the same degree of roughness exerts in a large canal or a river.

(7) That in the past canal builders have to a great extent overlooked the injurious effects of the growth of aquatic plants.

(8) That the effects of water plants in checking the flow and lessening the capacity of irrigation canals may be much greater than a rough, uneven channel.

(9) That in the sections of the arid West where such vegetation grows abundantly the canals should be built in such a way as to prevent its growth, or, if that is impracticable, provision should be made to facilitate its removal.

WOODEN FLUMES.

A half century ago the pioneers of Utah built flumes of native lumber to convey irrigating water across ravines or to strengthen weak places in canals in earth. Since that time their use in the Rocky Mountain States has steadily increased, and the cost of those at present in service would aggregate many million dollars. Notwithstanding their extended use they have not proved satisfactory. The short life of lumber when placed in contact with the soil and the difficulties

experienced in making wooden flumes water-tight have been a fruitful source of annoyance and expense and have brought such structures into disfavor.

During the last fifteen years many substitutes have been tried, such as stave pipes, half-round flumes, cemented canals, and inverted syphons, and although these later devices are more durable, as a rule they cost more, and it is questionable whether they will soon supplant the wooden flume. Like the railroad trestle, the wooden flume may be regarded as a temporary structure, but it possesses certain advantages which every canal proprietor or superintendent will appreciate. It is cheap, the materials of which it is composed can be readily and quickly procured, and the time occupied in building is short. So long as lumber is comparatively cheap and other building materials, such as Portland cement, are high, it is reasonable to conclude that the use of wooden flumes will be continued. The question for consideration is, therefore, the discovery of remedies for existing defects, rather than the construction of new structures to take the place of the old ones. The defects, as well as some of the remedies that might be applied, will be pointed out in the following discussion of the various features of the construction of wooden flumes.

FLUME LINING.

During the infancy of irrigation in the West flumes were put together in so unworkmanlike a manner that leaks were the rule and not the exception. The lumber was frequently of inferior quality, full of knots, unevenly sawed, and imperfectly dried. The lining consisted of 2-inch planks spiked to the posts and sills, and in less than a year the joints between the edges of the planks would be sufficiently open to admit the fingers of a man's hand. Laths and edgings were often driven into these open joints and afterwards they were calked with gunny sacks or oakum soaked in pitch. When the rough edges of the planks were beveled so as to leave a space at the joint having the form of a V, as shown in fig. 11, at *a*, it was possible to calk the joint, but if the edges sloped in the opposite direction and left the opening on the outside, as in fig. 11, at *b*, the calking would not remain in place.

Engineers and canal superintendents first tried to remedy these defects by putting on a double lining—an extravagant use of lumber. These linings were made by nailing battens over each longitudinal joint (fig. 11, *c*), by placing a 2-inch layer over inch boards (fig. 11, *d*), by using two layers of inch boards (fig. 11, *e*), or by using 2-inch planks beneath inch boards (fig. 11, *f*). In Montana 1-inch by 4-inch and 1-inch by 6-inch battens over joints are common, but they invariably are placed on the side next to the water, while in the opinion of the writer they should be placed on the outside. If, for example, battens were laid on the sills and the planks placed above them, the joints in

the latter would fill with sediment; besides, the action of the sun would not warp or curl the battens. If double linings of uneven

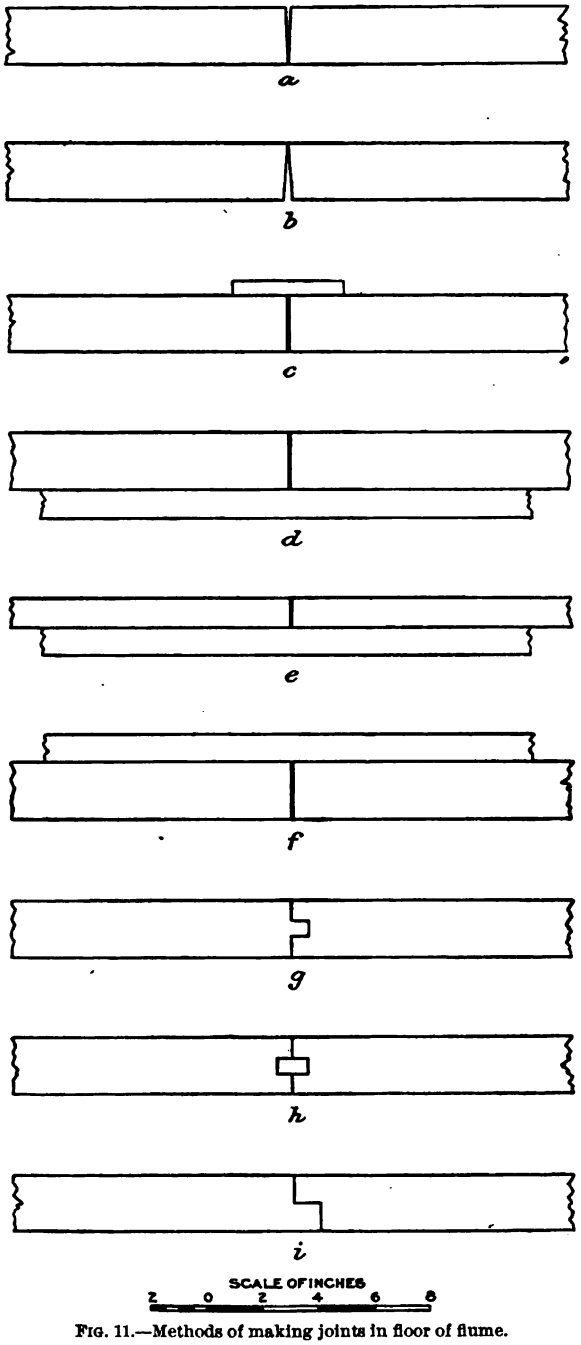


FIG. 11.—Methods of making joints in floor of flume.

thickness are used, it is preferable to lay the thinner layer first; otherwise the action of the sun will cause inch boards, particularly when

they become partially decayed, to draw through the nails. This same objection applies to the use of a double lining of inch boards.

The double-lined flumes of Colorado have not fulfilled the expectations of their builders. By using 2-inch planks over inch boards the cost is increased nearly 50 per cent and the durability is diminished. Experience has shown that a double lining will decay more rapidly than a single 2-inch lining. Besides, sand and sediment are likely to accumulate between the two floors, and to freeze in winter and cause displacement of the upper floor, so that the main object of the double floor—the prevention of leaks—is not always accomplished.

The common tongue and groove joint (fig. 11, *g*) has also been tried, but with little success. The cost of milling the edges of planks, the difficulty experienced in laying them, and the liability that the projections on each side of the groove will split off when the lining warps, have prevented this form of joint from being extensively used.

At the present time the tendency among irrigation engineers and superintendents is to demand a better grade of lumber for flume linings, to insist that the work shall be planned far enough in advance of construction to secure well-seasoned lumber, and to prepare the lumber at the mills in such a manner that the liability to leak may be either wholly prevented or greatly lessened. Only one lining is used, and its thickness may vary from 1½ to 2½ inches, depending on the size of the flume and other conditions.

The writer would recommend either the driven-tongue joint or the ship lap joint. The half-timbered or ship lap joint (fig. 11, *i*) is well adapted to linings varying in thickness from 1½ to 1¾ inches. It is true that it requires from 8 to 9 per cent more lumber to allow for the lap, besides the extra cost of milling; but notwithstanding this, the total cost is much less than that of a double lining, and if shrinkage occurs there is an excellent opportunity to calk the open space, in which case there is no chance for the filling to work through the joint. In the smaller flumes the edges are usually painted with white lead, and in the larger flumes with hot asphalt, both being applied during the process of construction.

The driven-tongue joint (fig. 11, *h*) requires thicker lumber; otherwise the projections at the edges of the groove would be likely to split off. A lining 2½ inches thick may have a groove on each edge five-eighths inch deep, which will admit a tongue five-eighths inch thick and seven-eighths inch wide. These tongues are milled from clear lumber and have the four sharp edges removed. The flooring is first laid, and as each plank is fitted to its place the tongue is inserted in the groove. No tongue is inserted in the last joint of the floor until after the planks have been matched and sprung into position, after which the tongue is driven into the groove from one end. The longitudinal joints in the sides of the lining are rendered water-tight

A LOWER END OF FLUME IN WEST CANAL BRANCH OF BEAR RIVER CANAL, UTAH

B. OLD FLUME OF DAVIS AND WEBER COUNTY CANAL COMPANY, WEBER CANYON, UTAH.

NORTH POUDRE FLUME, NEAR GREELEY, COLORADO, 300 FEET BELOW DAM, SHOWING CANAL CROSSING THE STREAM FROM WHICH ITS WATERS ARE TAKEN.

by driving wedges between the tiebeam and the upper edge of the lining.

FRAMEWORK.

Flumes are placed either on piles, on trestles, or on mudsills, and the form of the framework is modified to suit the foundation.

A cross section of a flume resting on piles is shown in fig. 12. Each set of piles is sawed off, after being driven, and capped with a dimension timber which is secured to each pile by means of a drift bolt. The stringers are then laid on the caps and the flooring is nailed to the stringers. The chief objection to this form of construction is the early decay of the piles at the surface of the ground. When thus exposed,

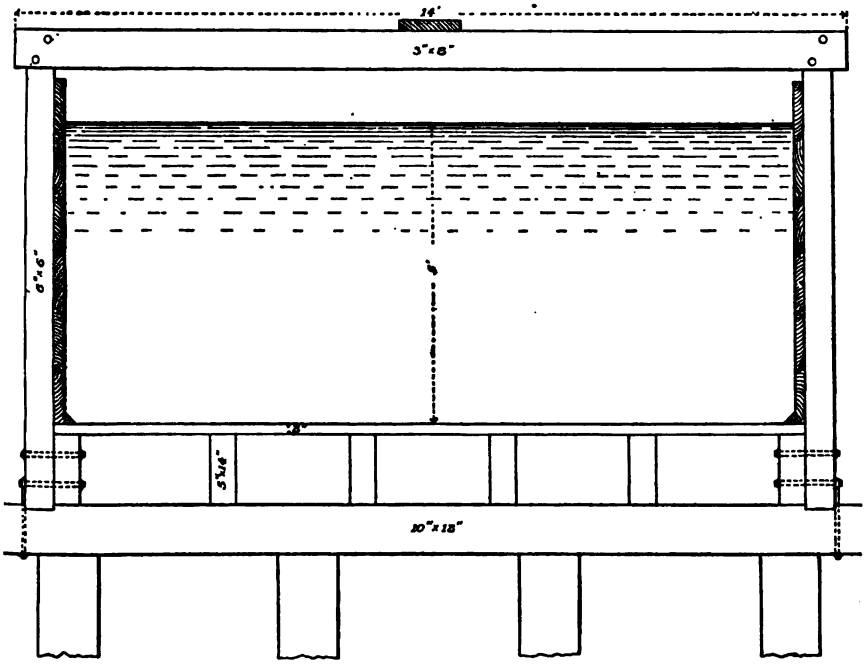


FIG. 12.—Cross section of flume resting on piles.

ordinary piles decay in about seven years. Frequently the top of the piles next to the cap is sound, as well as the portion that is covered with earth more than 2 feet in depth. In repairing flumes of this kind the practice of the writer has been to excavate the earth around each pile to a point beyond the decayed portion, remove the decayed wood, then recap the sound portion and introduce a trestle between the new cap and the flume.

When flumes are placed on trestles the ordinary railroad trestle is used. Small flumes, such as that shown in fig. 13, have only two inclined supports; in the larger flumes one or more vertical posts are inserted between the inclined posts. Sometimes farmers make use of

round poles to support flumes across ravines, in the manner shown in Pl. VII, *B*, which represents the Wilson canal flume, east of Ogden, Utah. As a rule, it is a mistake to make use of a wooden flume when

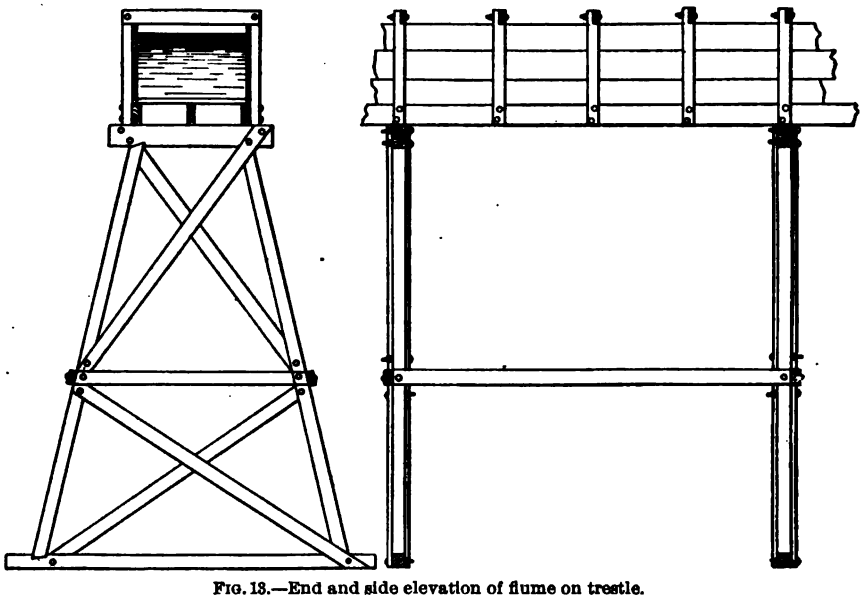


FIG. 13.—End and side elevation of flume on trestle.

trestles more than 10 feet high are required. Some other form of construction, such as the inverted siphon, should be adopted.

Very often portions of canals in earth, when located on steep hill-

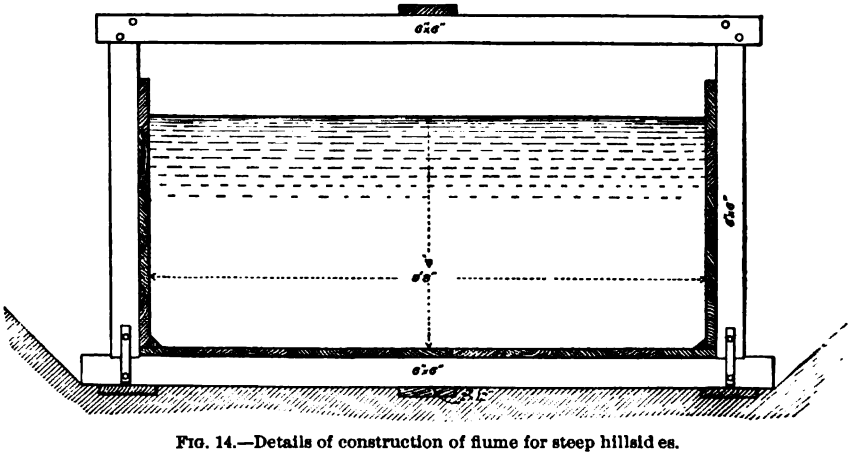


FIG. 14.—Details of construction of flume for steep hillsid es.

sides, absorb so much moisture as to become a source of danger to the canal and a menace to the property owners in the vicinity. In the past the canal in such weak places has usually been replaced by a

A. FLUME ON HEDGE CANAL, BITTERROOT VALLEY, MONTANA.

B. WILSON CANAL FLUME, WEBER COUNTY, UTAH

wooden flume somewhat similar in construction to that shown in fig. 14. The same kind of flume, with the exception of the mudsills, is frequently seen in rocky canyons, where a shelf has been blasted out to a sufficient width to support it. Occasionally, too, one sees, as in fig. 15, a flume which rests partly on rock and partly on timber supports. Fig. 15 is a cross section of the Dolores canal No. 2, in La Plata County, Colorado.

Flumes may also be divided into two classes, readily distinguishable by the manner of supporting the vertical posts. In the Rocky Moun-

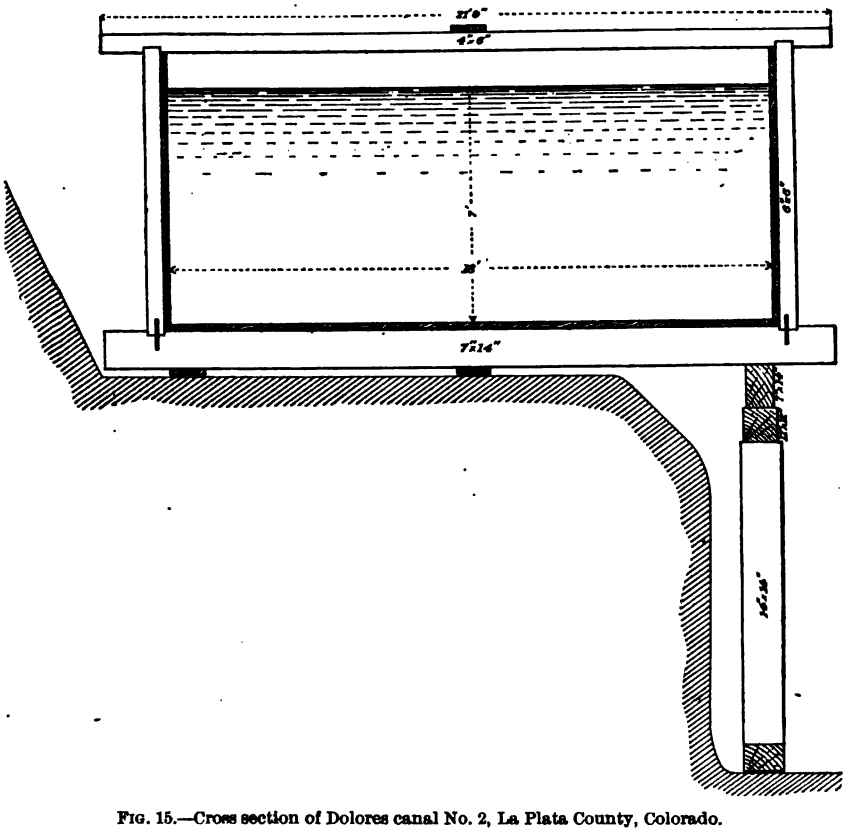


FIG. 15.—Cross section of Dolores canal No. 2, La Plata County, Colorado.

tain States the posts are usually held in position by a horizontal tiebeam bolted at its ends to a pair of vertical posts. In California it is customary to dispense with the tiebeam, and substitute therefor an inclined brace extending from the floor sill to a point in the upper half of each post. If one takes the trouble to compute the total amount of lumber required for each of these types, it will be found that the use of a tiebeam is more economical of lumber in all except the largest flumes. When a flume is more than 20 feet wide it may pay

to introduce inclined braces to secure the vertical posts; otherwise the horizontal tiebeams are preferable.

Fig. 16 shows a joint designed by the writer to connect tiebeams with vertical posts. This kind of joint leaves the tiebeam undimin-

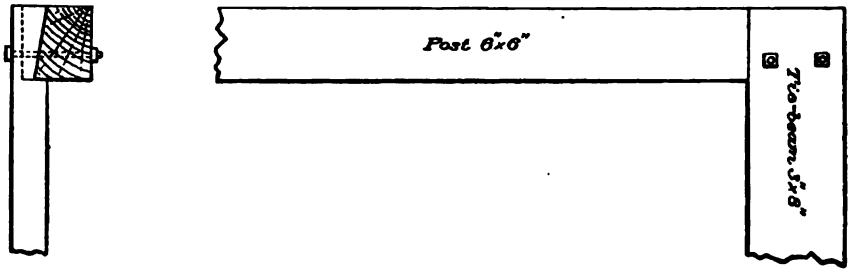


FIG. 16.—Joint designed to connect tiebeams with vertical posts.

ished in thickness at the ends, where it is liable to split and be cracked by the sun; it provides an ample shoulder to prevent wind or earth pressure from overturning the sides; and, being dovetailed, it is stronger than the common half-timbered joint.

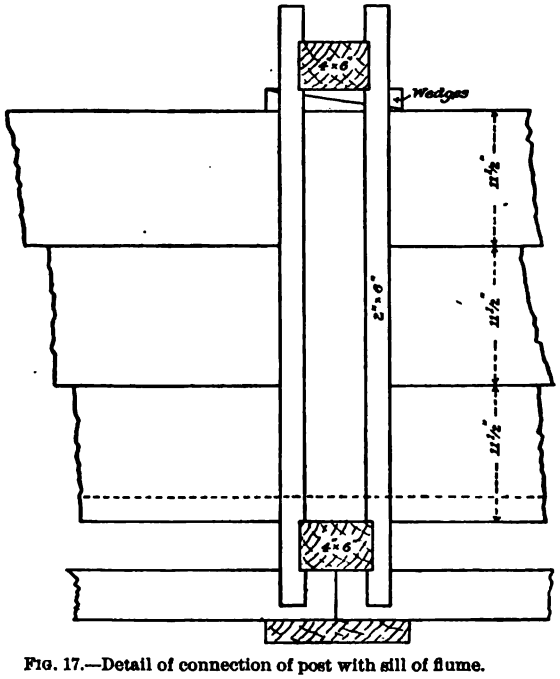


FIG. 17.—Detail of connection of post with sill of flume.

Mortise joints are now seldom used in building flumes; experience has shown that the sills first decay around the mortise. Besides, the holding power of a pinned-tenon joint in saturated pine wood is but slight. Vertical posts are now usually toe-nailed onto the sills, and if

there is any likelihood of the flooring settling away from the posts and sides, straps of steel should be bolted to each post and to the sill on which it rests, in a manner similar to that shown in fig. 14.

On the Bitterroot stock farm, owned by the late Mr. Marcus Daly and located near Hamilton, Montana, there are, in all, more than 5 miles of flumes on the various canals which supply water to that extensive ranch. These flumes are similar to those shown in Pl. VIII. They were designed by Mr. F. A. Jones, of Anaconda, Montana, chief engineer of the Anaconda Copper Company. The vertical posts, spaced 4 feet between centers, consist of two pieces of 2-inch by 6-inch joists, spiked with 40-penny wire nails to a 4-inch by 6-inch sill and tiebeam. The drawing shows a 3-inch by 6-inch tiebeam, but this was afterwards replaced by a 4-inch by 6-inch piece. These flumes are lined with 2½-inch plank, surfaced, edged, and grooved in the manner shown in Pl. VIII, and although built of green lumber they have always been practically water-tight. At the time of the writer's visit to this farm, in May, 1900, there were flumes of this design that did not leak one gallon per second in the length of a mile. In discussing this form of framework for flumes with Mr. Carlson, the carpenter and contractor who built them, he was of the opinion that the 2-inch by 6-inch posts should be gained into the sills and the tiebeams as shown in fig. 17. The following tables give a list of the lumber required for a 16-foot length of the flume and the dimensions of the standard trestle bents of various heights:

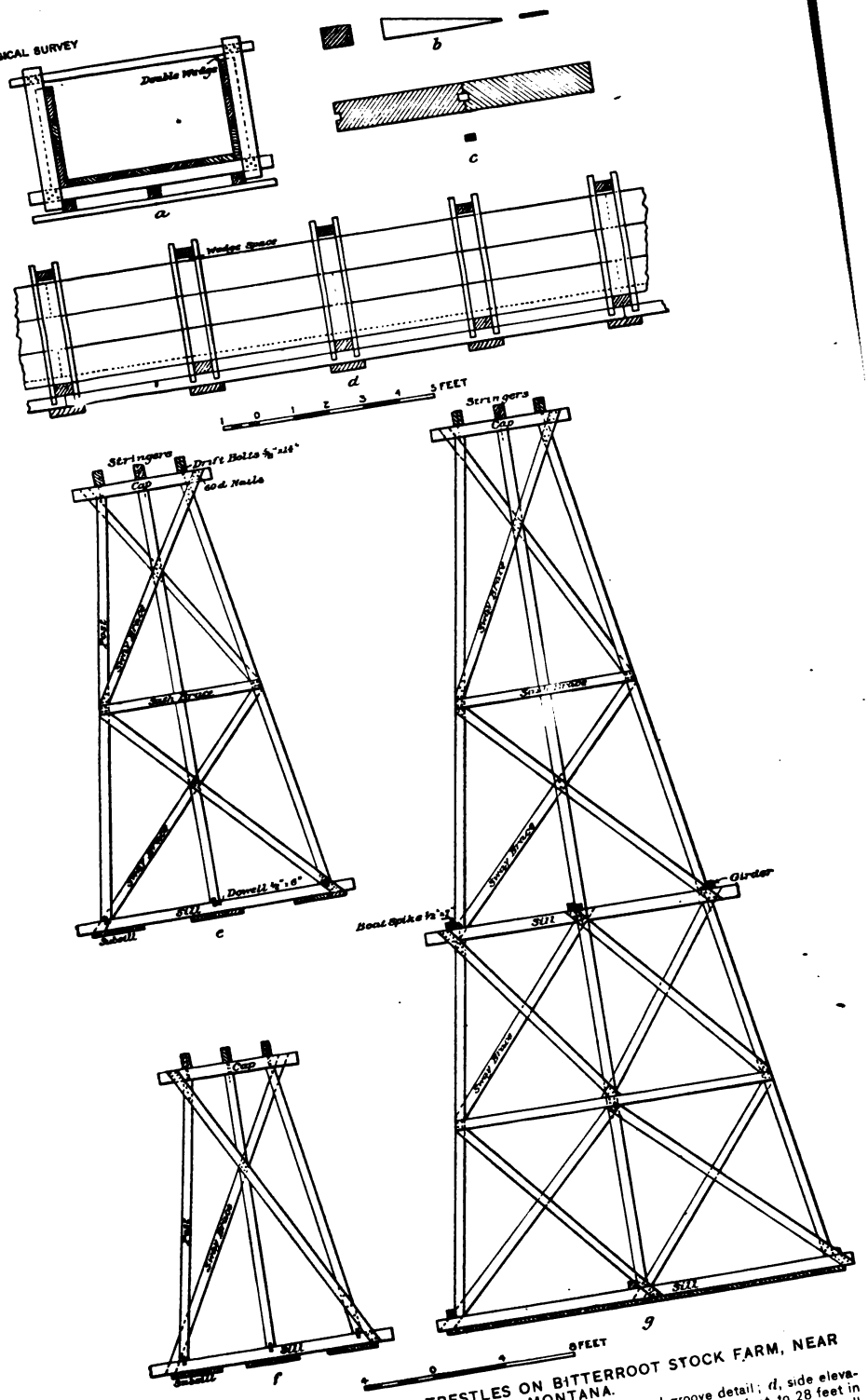
List of lumber required for a 16-foot length of 3-foot by 5-foot flume used in Skalkaho ditch, on Bitterroot stock farm, near Hamilton, Montana.

Parts.	Number of pieces.	Size.	Board measure.	Kind of lumber.
		Ins. Ins. Ft.	*Feet.*	
Sides and bottom	11	2½×12 ×16	440	Surfaced one side, both edges, and grooved.
Bottom yoke	4	4 × 6 × 7	56	Common rough.
Top yoke	4	3 × 6 × 7	42	Do.
Side yoke	16	2 × 6 × 4	64	Do.
Side-joint covers	4	3 × 6 × 3	18	Do.
Tongues	8	⅝ × ⅞×16	20	Clear first-class.
Wedges	16	{1¾ ¼}× 2½×10	5	Common rough.
Stringers	3	4 × 4 ×16	64	Do.
Subsills	4	2½ ×12 × 7	70	Do.
Total			779	

Dimensions of parts of standard trestle bents for 3-foot by 5-foot flume as used in Skalkaho ditch, on Bitterroot stock farm, near Hamilton, Montana.

Bottom of sill to top of stringer.		Plumb post, top of sill to bottom of cap.		Material for batter and plumb posts.	Sill.	Sway braces.	Sash.	Diagonal braces.
Ft.	*ins.*	*Ft.*	*ins.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*
3	8...	1	7½	2	8	.....	.....	.....
4	8...	2	7½	3	8	8	.....	.....
5	8...	3	7½	4	10	10	.....	.....
6	8...	4	7½	5	10	10	.....	.....
7	8...	5	7½	6	10	12	.....	.....
8	8...	6	7½	7	10	12	.....	.....
9	6...	7	5½	8	10	12	.....	18
10	6...	8	5½	9	12	14	.....	18
11	6...	9	5½	10	12	14	.....	20
12	6...	10	5½	11	12	14	.....	20
13	6...	11	5½	12	12	16	.....	20
14	4...	12	3½	13	12	16	.....	22
15	4...	13	3½	14	12	18	.....	22
16	4...	14	3½	15	14	18	.....	22
17	3...	15	2½	16	14	20	.....	22
18	3...	16	2½	17	14	12 and 14	10	24
19	3...	17	2½	18	14	14	10	24
20	3...	18	2½	19	14	14 and 16	10	26
21	2...	19	1½	20	14	14 and 16	10	26
22	2...	20	1½	21	16	14 and 16	10	20
23	2...	21	1½	22	16	16 and 18	10	20
24	2...	22	1½	23	16	16 and 18	10	20
25	2...	23	1½	24	16	16 and 18	10	22
26	1...	24	0½	25	16	16 and 18	10	22
27	1...	25	0½	26	16	16 and 18	10	22
29	4...	25 1	0½ 7½	26 2	16 and 18	16, 18, and 8–10	10	22
31	4...	25 3	0½ 7½	26 4	16 and 18	16, 18, and 10	10	22 and 18
33	4...	25 5	0½ 7½	26 6	16 and 18	16, 18, and 10–12	10	22 and 18
35	2...	25 7	0½ 5½	26 8	16 and 20	16, 18, and 12–14	10	22 and 18
37	2...	25 9	0½ 5½	26 10	16 and 20	16, 18, and 14	10	22 and 20
39	2...	25 11	0½ 5½	26 12	16 and 20	16, 18, and 14–16	10	22 and 20
41	0...	25 13	0½ 3½	26 14	16 and 20	16, 18, and 16–18	10	22
42	11...	25 15	0½ 2½	26 16	16 and 22	16, 18, and 18–20	10	22 and 24
44	11...	25 17	0½ 2½	26 18	16 and 22	16, 18, 12, and 12–14	10 and 18	22 and 26
46	10...	25 19	0½ 1½	26 20	16 and 22	16,18,12-14, and 14–16	10 and 18	22 and 20
48	10...	25 21	0½ 1½	26 22	16 and 24	16, 18, 14, and 14–16	10 and 18	22 and 20
50	10...	25 23	0½ 1½	26 24	16 and 24	16,18,14-16, and 16–18	10 and 18	22
52	9...	25 25	0½ 0½	26 26	16 and 24	16, 18, and 16–18	10 and 18	22

A common type of the small distributing flume is shown in fig. 18. The lining is usually 1¼ inches thick and 16 feet long. The bottom is



DETAILS OF FLUMES AND TRESTLES ON BITTERROOT STOCK FARM, NEAR HAMILTON, MONTANA.

At *a* is shown an end elevation of flume; *b*, wedge detail; *c*, tongue-and-groove detail; *d*, side elevation of flume; *f*, trestle bents for heights to 16 feet; *e*, trestle bents from 16 feet to 28 feet in height; *g*, trestle bents from 28 feet to 50 feet in height. Diagonal braces should be used on all [illegible] 10 feet up.

12 inches and the sides are 8 inches, with collars every 4 feet. The lumber is usually prepared at the mills, and each section of the flume is dipped in hot coal tar or asphalt as soon as completed. In the orchards of southern California such flumes are frequently used to distribute water to the various furrows, by means of 2-inch holes spaced midway between every two collars and controlled by a galvanized-iron slide. Fig. 19 illustrates a modification of these standard distributing flumes which was built by the writer to convey a small stream of water to experiment plats on the farm of the experiment station of Utah.

PROTECTION OF ENDS.

It is often difficult to make a secure and water-tight connection between the end of a flume and an earthen canal. Various devices

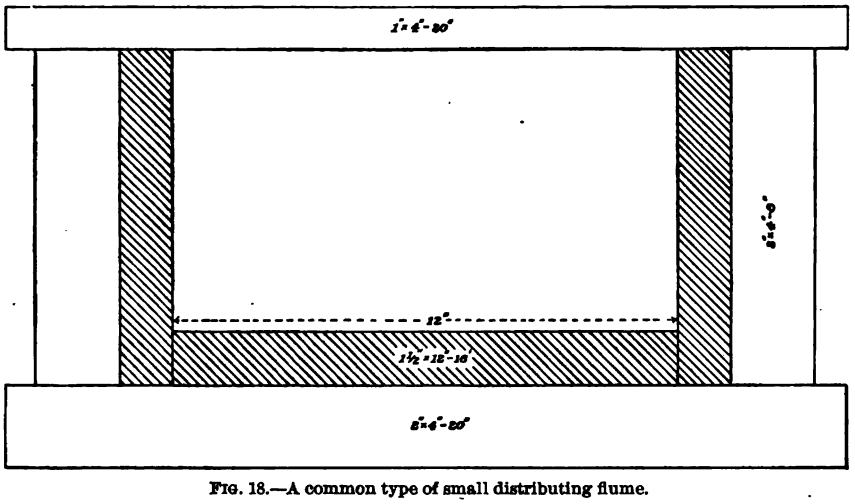


FIG. 18.—A common type of small distributing flume.

to accomplish this object have been used, but space will not permit more than a brief description of those most commonly adopted. This is an important feature of canal construction, for a large percentage of the breaks which have occurred are traceable to defective connections at the ends of the flumes. When flumes are built to convey water across ravines they should extend some distance beyond the edges of the ravines; otherwise a small leak, if unobserved, may wreck a costly structure.

Two forms of apron are used to prevent the passage of water either beneath or at the sides of a wooden flume, viz, the inclined apron and the vertical apron. The inclined apron extends downward from the floor, below grade, at an angle of about 20°, at which slope the lower ends of apron planks 10 feet long would be 3 feet below the bed of the canal. The side wings extend into the banks at even a greater angle,

and impervious earth is carefully tamped around both bottom and sides. This mode of protecting the ends is reasonably secure, but there are serious objections to it. In the first place, it requires a large

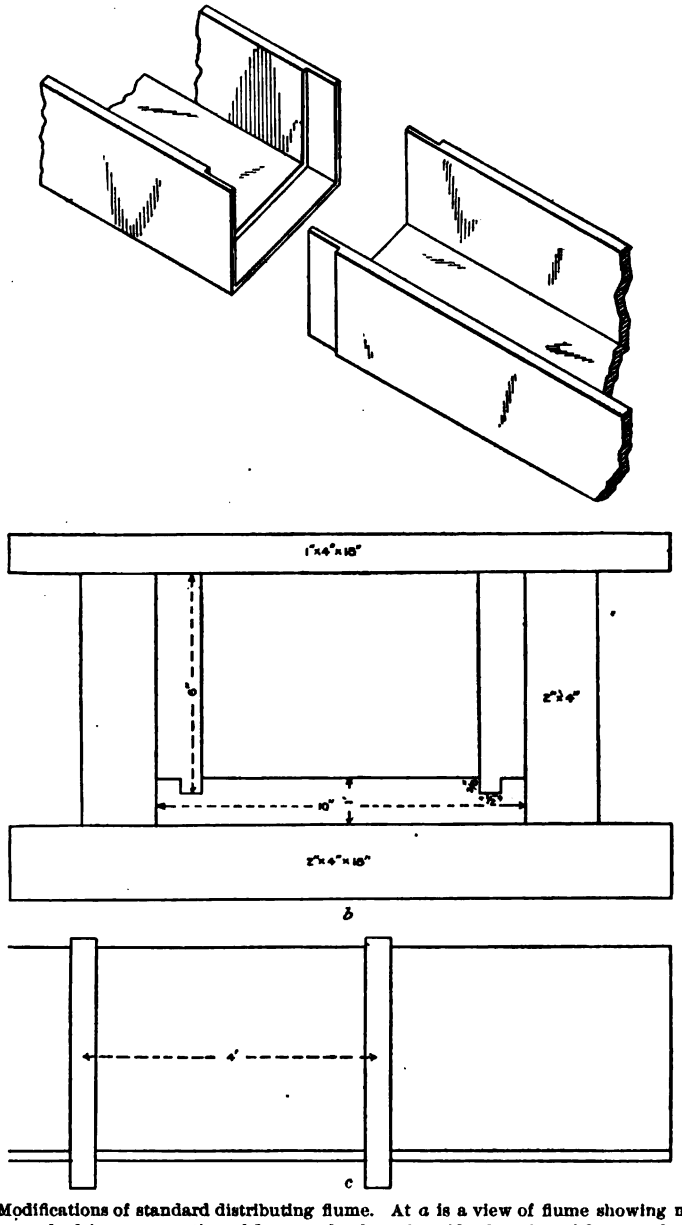
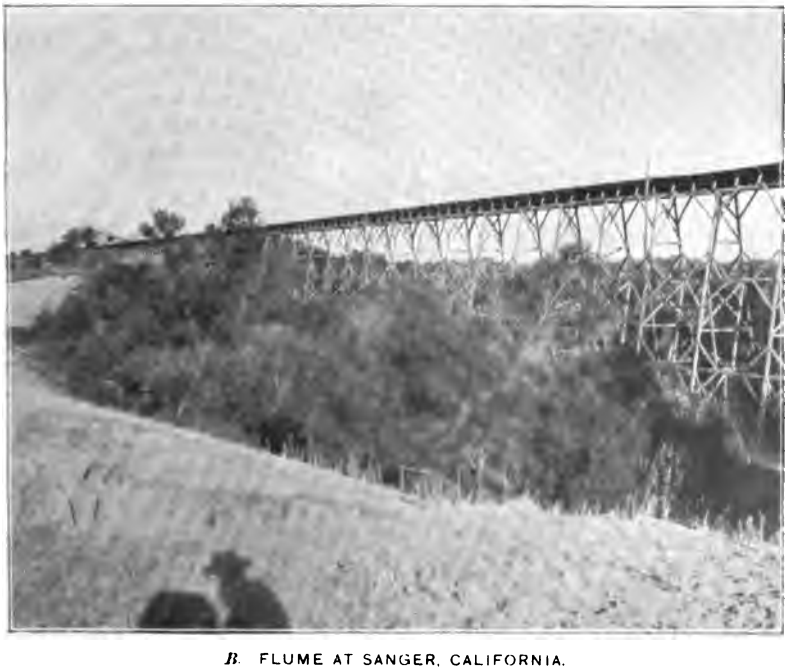


FIG. 19.—Modifications of standard distributing flume. At *a* is a view of flume showing method of joining ends; *b* is a cross section of flume and yoke; *c* is a side elevation of flume and yokes.

amount of material, and if we except the floor, which is below grade and always moist, this material, in the form of planking and dimension timbers, is placed in contact with earth and will decay in a few years.

A. FLUME OF KERN VALLEY POWER DEVELOPMENT WORKS, CALIFORNIA.



B. FLUME AT SANGER, CALIFORNIA.

There is another objection to the side wings: Every sudden change in cross section of water channels is likely to produce eddies. When water flows through a rectangular flume and the section is suddenly enlarged by wings placed at an angle of from 30° to 45° to the axis of the canal, the main body of water in the center flows on down the canal, but there is a part on each side which is deflected toward the banks and which forms eddies. These eddies are very destructive to the banks, and the form of construction which has a tendency to produce them should be avoided if practicable. Fig. 20 shows, in plan and elevation, the end of a flume with inclined aprons. When protected by riprap the vertical apron possesses certain advantages over

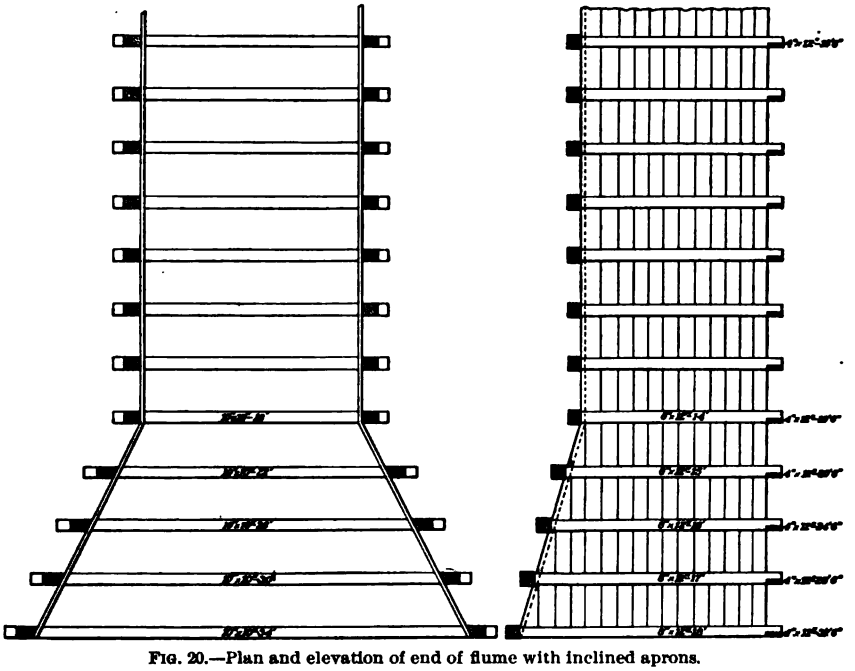


FIG. 20.—Plan and elevation of end of flume with inclined aprons.

the inclined apron. At the end of the flume a trench is dug 3 or more feet below grade and extending beyond the flume on each side to a distance of about half the width of the flume. An additional mudsill is laid in the bottom of the trench, and additional posts are placed at the ends, with an extended tiebeam on top. Planks are then spiked to these timbers and the earth is carefully tamped around them. This makes a water-tight bulkhead around each end of the flume. Since the amount of lumber required for this kind of apron is small compared with that required for the inclined apron, the writer has frequently obtained California redwood for the purpose. Pine lumber, if treated by some of the well-known and now much-used preservative processes, would, however, answer as well. To prevent erosion and the formation of eddies, riprap should be placed on the bottom and

sides, in the manner shown in fig. 21. If care is used in riprapping the sides, it is possible to so gradually change the cross section of the water in the flume to that of the canal that the flow will be but slightly disturbed.

There is a modification of the vertical apron introduced by Mr. J. C. Ulrich, civil engineer, of Colorado,[1] which is deserving of mention. In addition to the vertical apron, Mr. Ulrich builds at each end a short flume which he places 3 or more feet below grade and fills the additional space thus made with well-tamped earth.

CARRYING CAPACITIES.

As a rule the cross sections of flumes are too small. In calculating the size of a flume for a canal it is customary to take a high coefficient

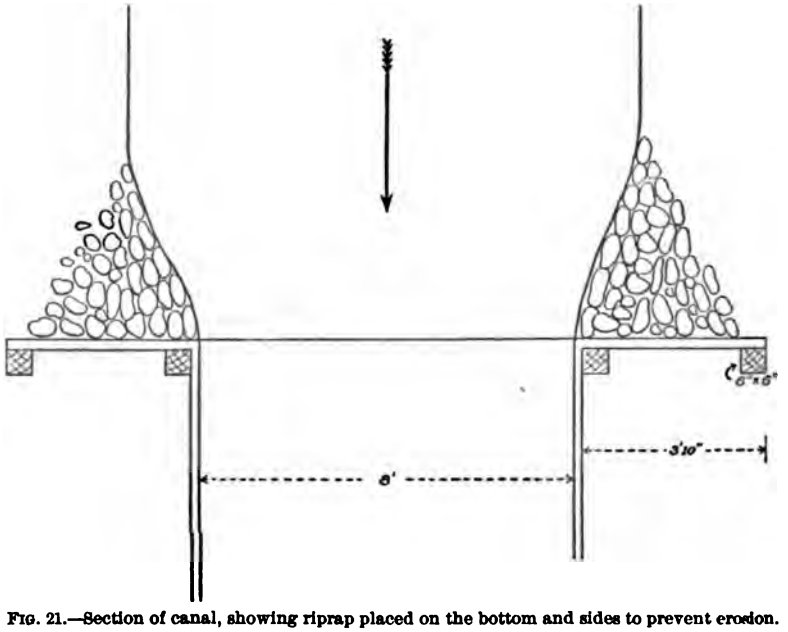


FIG. 21.—Section of canal, showing riprap placed on the bottom and sides to prevent erosion.

of roughness for the canal and a low coefficient of roughness for the flume, for the reason that the flume is lined with lumber and the canal is composed of earth. The result of such theorizing is to overtax the upper ends of the flumes. In reality there is little difference between the carrying capacity of the flume as ordinarily built and the carrying capacity of an earthen canal of the same sectional area. All of the shorter flumes have their floors covered with sediment, gravel, and occasionally with cobbles, and the friction in them, if the lining be rough, may be greater than in the ordinary canal.

When a flume is built on the same grade as the portion of the canal

[1] See discussion on flumes in Ann. Am. Soc. Irrig. Engrs. for 1892-93, p. 181.

adjacent to it, its width in the clear should be about equal to the mean width of the bottom and the surface of the canal; that is to say, if a canal has a bottom width of 10 feet and a surface width of 16 feet, the proper width of a flume for such a canal would be nearly 13 feet in the clear.

When water is to be conveyed through a flume crossing a ravine several hundred feet in width, it is customary to allow an extra fall for the flume, in order to reduce its size and lessen the expense. This extra fall can not be utilized to advantage if the size of the flume is uniform throughout, since the water moves with ordinary speed as it enters the upper end, and the effect of a steep grade is to increase the velocity and diminish the depth, so that the flume at the lower end may not run half full. In order to utilize an extra fall of this kind, either the width or the depth of the flume should be gradually diminished from the upper end. By making the upper end as wide and as deep as the water in the canal, the width, for example, may be gradually lessened as the velocity of the water increases. This would make the most economical flume as regards material, but the cost of construction would be increased, which may account for the fact that few flumes have been built in that way. Some of these disadvantages may be overcome, however, by gradually enlarging the upper part of the flume, and allowing a steeper grade for this tapering part. By this means the entire volume would be admitted without damming back the water, and the extra grade would increase the velocity sufficiently to allow the flume proper to convey the volume admitted.

The following experiments were made on the carrying capacities of flumes:

Experiment No. 23.—This test was on a flume of the Bear River canal at the lower Malade River crossing. About 200 feet of the upper portion is of iron and steel, lined with riveted plates; the lower portion, for a distance of more than 125 feet, is of wood. The test was made in the latter portion, and the results are given in the table on page 60. The discharge, 64.34 second-feet, was obtained by current-meter measurement. There was no sediment or gravel in the bottom, and the planks which formed the lining were planed.

Experiment No. 45.—This test was on a small flume on the Elm farm, near Corinne, Utah. The lumber forming the bottom and sides was planed, but some of the oakum with which the joints were calked projected in places. The discharge was measured over a trapezoidal weir.

Experiment No. 8.—This test was on a large flume of the Bear River canal, located near the mouth of Bear River Canyon. The lining was of planed lumber, but the floor of the flume was almost entirely covered with sediment and fragments of soft rock. Three current-meter measurements were made at the top, middle, and

bottom of a 150-foot section, and the results were 197.40, 197.32, and 197.78 second-feet, respectively.

Experiment No. 10.—This test was on a large flume of the Bear River canal, located about 2 miles above that of the flume of experiment No. 8. In this case the floor was covered with sediment, gravel, and small cobbles. The results of the current-meter measurements, taken at the top, middle, and bottom of a 100-foot section, were 206.36, 207.74, and 206.89 second-feet, respectively.

Table showing values of hydraulic elements in experiments to determine carrying capacities of flumes.

Experiment.	Discharge.	Area of water section.	Mean velocity.	Hydraulic mean radius.	Wetted perimeter.	Coefficient of roughness (n).	Slope.	Coefficient c.*
	Sec.-feet.	*Sq. feet.*	*Feet per second.*		*Lineal feet.*		*Feet per foot.*	
No. 23..	64.34	11.78	5.46	0.87	13.62	0.0142	0.0032	103.37
No. 45..	0.97	1.49	0.65	0.33	4.51	0.0184	0.00038	57.64
No. 8..	197.50	66.60	2.97	2.86	23.32	0.0201	0.0004	87.72
No. 10..	207.00	84.05	2.46	2.94	28.60	0.0217	0.00031	81.85

*In Chezy's formula.

SEMICIRCULAR FLUMES.

This form of flume, which resembles the modern stave pipe more than the rectangular flume, was invented by Guy Sterling, civil engineer, nearly nine years ago. His claims for a patent were filed in the summer of 1892, and in August, 1893, a patent was issued. The first flume of this type was built by William Ham. Hall, chief engineer of the Santa Ana canal, near Redlands, California, from a small model and a rough sketch prepared by Mr. Sterling. Inasmuch as the Santa Ana flume differs in many particulars from those built under the Sterling patent in Utah, it is reasonable to conclude that Mr. Hall modified Mr. Sterling's design. In this flume the 8-foot staves are milled from 2-inch by 6-inch joists, in a manner exactly similar to the staves for wooden pipes, and are supported at the ends by T irons curved to coincide with the exterior of the flume. These T irons rest on wooden sills which are concave in the center and are supported by either blocks of concrete or by redwood mudsills. Each section of the flume is also bound by two five-eighths-inch round steel rods, fastened by means of nuts to two horizontal tiebeams which rest on the top edges of the flume. At their ends the staves are tightened by means of wedges driven between the tiebeams and iron straps, and throughout the middle portion they are tightened by cinching the nuts onto the two iron rods.[1]

[1] See Trans. Am. Soc. Civ. Engrs., Vol. XXXIII, p. 100.

END VIEW OF STERLING FLUME IN PROVO CANYON, UTAH.

SIDE VIEW OF STERLING FLUME IN PROVO CANYON, UTAH.

At least four lines of this type of flume are now in successful operation. Their lengths, areas, and locations are as follows:

List of Sterling patent semicircular flumes in operation.

Location.	Length.	Sectional area.
	Lineal ft.	*Square ft.*
Santa Ana canal, Redlands, California	11,394	15.50
Mount Nebo Irrigation Co., Goshen, Utah	5,800	14.14
Utah mining camp	500	8.00
Telluride Power Transmission Co., Provo, Utah	8,400	28.00

With the exception of the Santa Ana, all of the flumes named have been built in accordance with one general plan, differing only in size and relative proportions. The Mount Nebo Irrigation Company's flume is a true semicircle of 6 feet inside diameter; the mining camp flume resembles a parabola, its depth being about 4 feet and its greatest width about 3 feet; and the Telluride Power Transmission Company's flume is shown in fig. 22. The staves are connected at their ends by metallic tongues inserted in saw kerfs, and are broken-jointed in that every two adjacent staves overlap at least 2 feet. The form of the staves, the end joints, and the manner of breaking joints are thus similar to those of stave pipe. The semicircular flume rests on wooden chairs placed 10 to 12 feet between centers. Each chair consists of a bolster, two vertical pieces curved by band sawing to suit the exterior of the flume, and a straight tiebeam. The chairs for the large flume of the Telluride Power Transmission Company in Provo Canyon are placed 12 feet apart, and the half-inch steel bands which hold the staves together are spaced 2 feet 8 inches between centers.

The Sterling semicircular flume possesses certain advantages over the common rectangular flume. In canyons where the line of the flume is made up for the most part of curves, it is comparatively easy to adjust the Sterling flume to ordinary curvatures. In case of shrinkage the flume may readily be made water-tight by screwing up the nuts at the ends of each threaded band. It also possesses the advantage of not being in contact with the soil, so that air is allowed to circulate freely around all portions, with the exception of the small mudsills, which may be replaced at trifling cost. No nails or spikes are driven through the lining of the flume, and by making a girder of each 12-foot length fewer supports and less lumber are required.

The Provo Canyon line, shown in Pls. X and XI, has been successfully operated for nearly three years, and the only breaks that have occurred have been caused by bowlders becoming detached from the canyon sides and striking the exposed flume. The bottom staves

sagged in places to such an extent that additional supports had to be inserted midway between the chairs. This is the most serious structural weakness which has developed in this flume, and the remedy is evidently a shorter span between the chairs.

In order to determine the carrying capacities of semicircular flumes, two current-meter measurements were made in 1898 by the late W. B. Dougall, civil engineer, who was then assisting the writer. One of these measurements was made near the intake, in what is known as the

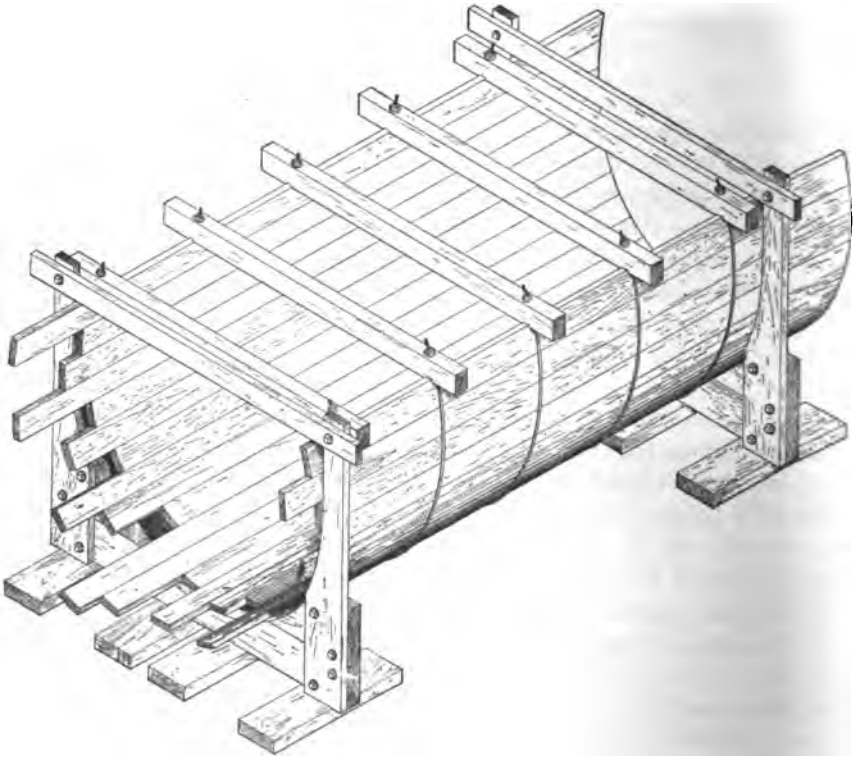


FIG. 22.—Flume of Telluride Power Transmission Company in Provo Canyon, Utah.

old flume, and the other was made at a point about a mile below, in what is known as the new flume. At the time these experiments were made the flume was not carrying its maximum quantity of water, for some repairs were being made. There was also considerable leakage between the two places of measurement, which may account for the greater part of the difference in discharge. The following are the results of Mr. Dougall's experiments:

A. TEMPORARY HEADWORKS OF SANTA ANA CANAL, CALIFORNIA.

B. CONNECTION BETWEEN TUNNEL AND FLUME OF SANTA ANA CANAL, CALIFORNIA.

Table showing results of tests made by W. B. Dougall to determine the carrying capacities of semicircular flumes.

Measurement.	Discharge.	Area of water section.	Mean velocity.	Hydraulic mean radius.	Wetted perimeter.	Coefficient of roughness (n).	Slope.	Coefficient.*
	Sec.-feet.	*Sq. feet.*	*Feet per second.*		*Lineal feet.*		*Feet per foot.*	
Upper measurement.	74.39	13.24	5.62	1.45	9.12	0.0111	0.001	147.74
Lower measurement.	72.40	13.49	5.37	1.46	9.26	0.0116	0.001	140.84

* In Chezy's formula.

STAVE PIPE.

The use of wood to convey water for domestic purposes was common a hundred years ago. Many of the present extensive and costly waterworks systems of the New England cities had their origin in one or more lines of bored logs. Owing, however, to their small capacity and the imperfect manner of joining the lengths, their use in the United States has been wholly superseded by some form of metal pipe. One frequently reads of these old bored logs being dug up to give place to modern improvements, and the excellent state of preservation in which they are found, after being buried a lifetime, speaks well for the lasting qualities of wood under favorable conditions.

FIG. 23.—Water pipe of bored logs.

In some parts of Canada bored logs are still extensively used for water pipes, on account of the cheapness of suitable lumber. Tamarack wood is preferred, both on account of its durability when in contact with the soil and the suitable form of the trees, which are frequently not larger than a stovepipe at the butt end and maintain a nearly uniform size throughout two-thirds of their entire length. When tamarack is not available, Canada balsam makes a good substitute. In many sections this can be had for the cutting. It is so soft that a man can bore with ease a 2-inch hole through a section length, and when kept moist it lasts well. The trees are cut in lengths of from 9 to 12 feet, and each length is placed on a platform, where it is bored by a long auger revolving in guides and operated by hand. The sections are joined by reaming a long, tapering socket in the larger end of each length and by shaving or turning the smaller end into a spigot. Formerly a band of wrought iron encircled the socket end to prevent the shell from splitting when

the spigot was being driven into it; but in more recent practice the iron band is driven into the end, as shown in fig. 23.

Illustrations of the next stage of the development of wooden pipe are to be seen in the penstocks of many of the old gristmills and sawmills of New England and eastern Canada. Originally these pipes, or penstocks, were built in sections of narrow pine or balsam planks, the edges being planed, by hand, to the required bevel. These staves were encircled by flat bands, to which were welded short pieces of round iron threaded and secured by nuts to cast-iron saddles. The weak feature of this construction was in the joints between the sections. By using a tapering stave the ends of each section were made to differ in size, and by shaving down the smaller end it was made to telescope the larger. The lap, however, was only a few inches at most, so that the joints usually leaked, or the sections slipped apart altogether. In some cases this defect was remedied by building the pipe in the trench

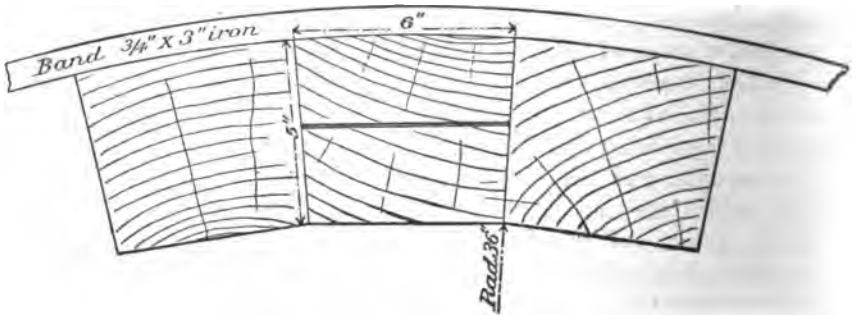


FIG. 24.—Section of stave pipe used in wooden conduit of Toronto waterworks.

and avoiding distinct joints at regular distances, the ends of the staves being united by an iron dowel and the joints broken by a lap of 2 or more feet.[1] The 6-foot wooden conduit laid in 1881 and 1882 as part of the intake of the Toronto waterworks may be cited as a sample of this type of stave pipe. The staves were made of white oak 6 inches wide and 5 inches thick, beveled on the edges and curved on the outer surface to conform to an arc having a radius of 3 feet 5 inches, as shown in fig. 24. The metallic tongue between abutting stave ends was one-eighth inch thick, 6 inches long, and the full width of the center of the stave. The hoops were of wrought iron, three-fourths inch thick by 3 inches wide, welded together, washtub fashion, and spaced 2 feet between centers.

It is a modified form of this kind of stave pipe that has been so extensively used in the Western States during the last twelve years.[2] The first line of this kind was built under the supervision of Mr.

[1] See paper by T. J. McMinn in Trans. Can. Soc. Civ. Engrs., Vol. I, Pt. I, p. 67.
[2] Wooden-stave pipe, by Samuel Fortier: Ann. Am. Soc. Irrig. Engrs. for 1892-93.

STAVE PIPE OF ELECTRIC-LIGHT WORKS AT BRIGHAM, UTAH.

Charles P. Allen, chief engineer of the Denver Union Water Company, of Denver, Colorado, and extended from the West Denver reservoir to the underground galleries on South Platte River, 3½ miles distant.

The results of Mr. Allen's experiments were finally embodied in a patent issued March 22, 1887. This patent embraced stave pipe as a whole as well as each of its main features, viz, the round iron or steel bands, the cast-iron shoes or saddles, and the metallic tongues. Many modifications and some improvements, now covered by other patents, have since been made, and in order that the reader may obtain some idea of the modern forms of this pipe, a brief description of each will be given.

Fig. 25 illustrates the mode of connecting the butt joints of the

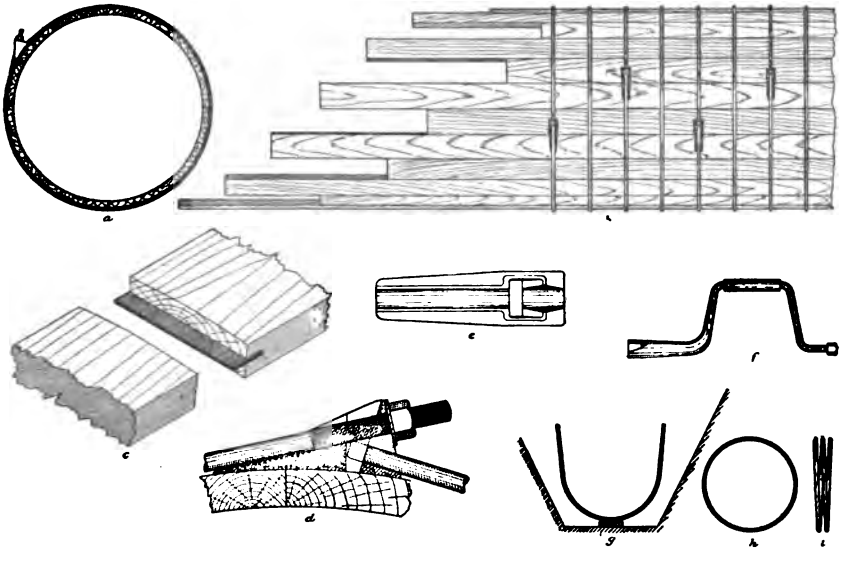


FIG. 25.—Details of Allen type of stave pipe. *a*, section of wooden pipe, showing position of steel bands and iron saddle; *b*, elevation of pipe, showing method of breaking joints and position of steel bands; *c*, ends of staves, showing metallic tongue; *d*, ends of steel band and malleable-iron saddle; *e*, plan of saddle; *f*, wrench or bitstock for screwing up nuts on steel bands; *g*, outside form used in building pipe, consisting of a bent gas pipe on a block of wood in the trench; *h* and *i*, inside form used in building pipe, consisting of a coil of ¾-inch gas pipe.

staves, the forms of the steel bands and the malleable iron saddles, and a portion of a completed pipe of the Allen type. The staves are dressed from joists varying in size from 2 inches by 4 inches to 3 inches by 10 inches, depending on the diameter of the pipe to be built, and their ends are slotted to admit the metallic tongue. The malleable-iron saddle is so designed that it holds in position the T end of the band, and allows the insertion of the threaded end, which is secured by means of a nut.

Fig. 26 shows the construction of the Dwelle type of stave pipe. It differs from the Allen type in several important particulars: The

ends of the staves are connected by means of a tongue and groove; the band is threaded at each end and is secured to the casting by two corresponding nuts which abut against shoulders; the edges of the staves, instead of being radial, are dressed to a kind of ogee curve.

Fig. 27 shows the construction of the stave pipe recently built by the Pioneer Electric Power Company, of Ogden, Utah. The shape of the staves and the metallic tongues used to connect their ends are similar to those of Allen's patent, but the steel bands and saddles are widely different; they were designed by the late J. C. O'Melveny, civil engineer, of Salt Lake, Utah. The band is in two parts. One part encircles the upper circumference of the pipe and is attached to two steel saddles by means of two loop eyes; the other part encircles the lower circumference of the pipe and is attached to the same sad-

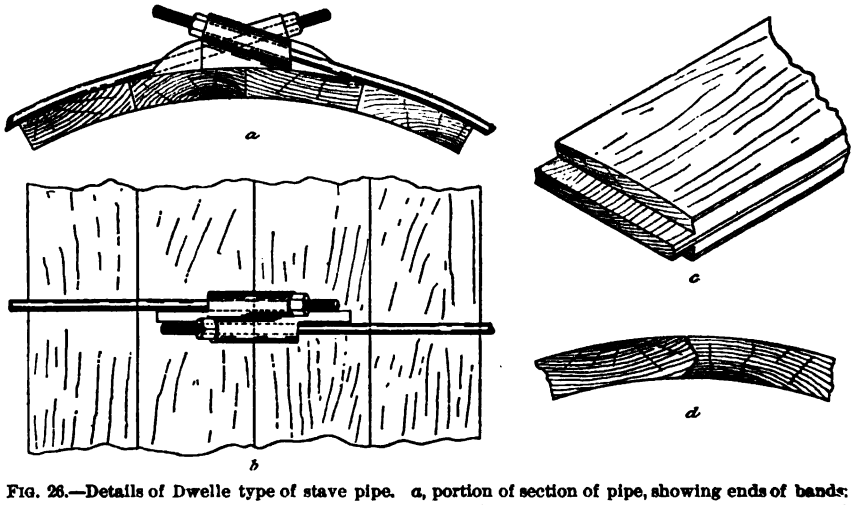


FIG. 26.—Details of Dwelle type of stave pipe. *a*, portion of section of pipe, showing ends of bands; *b*, plan of pipe; *c*, end of stave, showing tongue; *d*, portion of section, showing edges of stave dressed to ogee curve.

dles by means of nuts screwed to each threaded and upset end of this part of the band. The saddles are U shaped, being curved to suit the outer arc of the staves, and pressed, when hot, out of three-sixtenth-inch steel plates.

There are various other modifications of staves, bands, and saddles, but their use has been confined to short lengths of pipes and to few localities.

LUMBER FOR STAVES.

Colorado pine (*Pinus ponderosa*) was used on the first pipe lines built in the West. Its tendency to warp, the large number of knots which it contains, and its short life when exposed to unfavorable conditions, render it an unsuitable wood in every respect save that of first cost.

A. OLD FLUME AND NEW REDWOOD STAVE PIPE REPLACING IT, REDLANDS CANAL, CALIFORNIA.

B. REDWOOD STAVE PIPE UNDER 160 FEET HEAD, SANTA ANA CANAL, CALIFORNIA.

Long-leaf yellow Texas pine (*Pinus palustris*) was next tried and was found to possess many advantages over the native pine. There were few knots, the staves were longer, and there was less warping on exposure to the action of the sun and atmosphere. The resin-filled ducts in the quarter-sawed lumber were among the worst defects, since the fluctuating water pressure within the pipe washed out the resinous substance and allowed the water within to ooze through the thin shell.

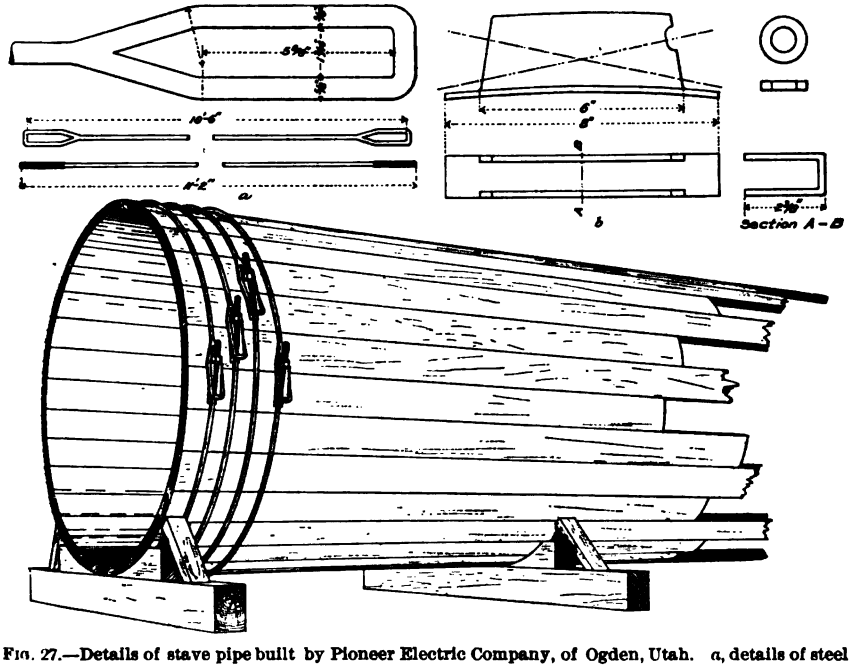


FIG. 27.—Details of stave pipe built by Pioneer Electric Company, of Ogden, Utah. *a*, details of steel bands; *b*, plan and elevation of saddle.

California redwood (*Sequoia sempervirens*), on account of its great durability, its freedom from knots, and its fine, close texture, has been extensively used for this purpose on the Pacific coast and as far east as Utah, Montana, and Colorado. More than twenty carloads of finished redwood staves were used by the writer in the spring of 1890 to build a 2-foot stave conduit for the Ogden waterworks, and two years later the same kind of wood was used on the 18-inch conduit of the Logan waterworks. Both of these lines have given excellent satisfaction and bid fair to last thirty years or longer.

Douglas spruce (*Pseudotsuga douglasii*), commonly called Oregon fir, or Oregon pine, was used to the extent of 1,500,000 feet B. M. on the 27,000 lineal feet of 6-foot conduit of the Pioneer Electric Power Company, of Ogden, Utah. In appearance this lumber was perhaps

the finest ever used in construction work on this continent. It was strong and smooth, and entirely free from knots and other defects common to most timbers. The only question yet to be settled in regard to the use of Douglas spruce for staves is: Will it last long? We know that the life of a redwood railroad tie, if protected by thin metal plates from the crushing effect of the rail, is more than twice as long as that of either a pine or a spruce tie. The greater part of the conduit just mentioned is considerably below the level to which the water would rise if a hole were bored in the pipe, and the lumber in all such portions will be continuously soaked. Such being the case, the lumber used was perhaps the most economical. If, however, a pipe or conduit has to be located on grade, or in a position where the pipe will be only partially filled with water, there can be no question that redwood, although its first cost may be greater, will prove the cheapest wood in the end.

STEEL FOR BANDS.

In specifying the quality of steel for the bands, the strength of the metal has been considerably reduced in order to increase its toughness and ductility. Mr. Allen was the first to use a mild steel made by the open-hearth process, very similar in quality to that used for steam boilers. The average tensile strength of the specimens tested was about 61,000 pounds per square inch, the elastic limit 40,000 pounds, the elongation in 8 inches more than 25 per cent, and the reduction area more than 50 per cent. This kind of steel will not only bend flat upon itself through 180 degrees when cold, but after being so bent it may be hammered to half its diameter without sign of fracture.

In the first lines built in the West the rods were not upset at their threaded ends. This omission was probably due to the extra cost of upsetting small rods. This work can now be done, however, for a mere trifle, and any one can readily see that in pipes of large diameters, requiring long bands, the saving in upsetting the threaded ends is considerable.

CONSTRUCTION.

The bottom width of the trench is made from 1 to 2 feet wider than the diameter of the pipe, in order to give space for workmen to stand on either side of the pipe when it is being put together, and the side slopes of the trench are as steep as the nature of the material will permit.

Ordinarily inch gas pipes bent in the form of a U, the lower portion conforming to the outer circumference of the pipe, answer very well for outside forms in which to hold the staves in place until the lower half is put together. To prevent the staves from falling while the upper half is being built, the same kind of iron pipe is used in the

REDWOOD STAVE PIPE (52-INCH) CROSSING WARMSPRINGS CANYON, NEAR REDLANDS, CALIFORNIA

form of a coil, the outer diameter of which is nearly equal to that of the inside diameter of the pipe. The bottom staves are kept slightly in advance of the top staves, in order to facilitate building and to make a stronger joint. The lengths are joined by lapping the staves a distance of from 20 to 30 inches. An oak driving bar banded with Norway iron and struck with a 12-pound sledge hammer is the only device used to drive the staves and make tight all end joints. The metallic tongues are inserted in the slots made in the ends of the staves after each section or length has been driven, when all is in readiness for the building of the next section.

The steel rods for the bands are bent by hand after being shaped on turning tables, and are afterwards either dipped in hot asphalt or painted. Specially designed braces are used to screw the nuts on the bands, and a leather-faced wooden mallet handled by a man inside of the pipe has been found to be the best tool to adjust the staves to the proper alignment and true circle.

The strength of the pipe depends on the number and size of the steel bands. If there is little pressure within the pipe, the bands are spaced far apart, but seldom more than 14 inches. As the pressure within increases—i. e., as the pipe dips—the bands are placed nearer together, so that a pipe under a head of 100 feet may be as strong as a pipe under a head of 5 feet.

LOCATION OF PIPE LINE.

It is very bad practice to locate a wooden pipe line on a grade, for the reason that when so placed the pipe is seldom full of water, and moist air and gases collect in the upper portion, which, with the earth without, soon cause the wood to rot. An ideal location is over gently undulating ground where each low, flat valley ends in a narrow ridge. Seen from the side, a pipe so placed would remind one of a big cable supported at regular intervals and sagging between every two supports. Hence, in laying out a line for wooden pipe, no matter how rough or uneven the country traversed may be, an attempt should be made to rise to grade only at intervals which might vary from a quarter to a half mile, while the line between the rising points might be 5 or more feet below grade. The main object of such a location is to keep the pipe not only full of water but under some internal pressure. Other conditions being favorable, and the shell being kept continuously moist, wooden pipe should last a lifetime.

DURABILITY.

Stave pipe built of native pine lumber, placed in a dry soil, and kept only partially full of water, may rot along the top staves to

such an extent that in five years those staves will have to be renewed. The same kind of pipe, however, if kept continuously full of water and under pressure and buried 2 or 3 feet beneath the surface, may be in good condition twenty years after being laid.

Ten years ago the city of Cheyenne, Wyoming, laid about 9,000 lineal feet of stave pipe from wells in Crow Creek to a standpipe near the reservoir. The low standpipe kept the lower portion of the pipe line under pressure, but the upper portion was seldom full of water, often only half full, and rotted so rapidly that in about eight years 2,000 feet of it was replaced by sewer pipe, and during the year 1897 an additional length of 2,200 feet was replaced by sewer pipe 24 inches in diameter. Through the courtesy of Walter D. Pease, civil engineer, of Laramie, Wyoming, the writer obtained three samples of the old staves, which Mr. Pease graded best, second best, and third best. The best sample was in fairly good condition, having been kept moist, the third-best sample was greatly decayed, while portions of the top staves were reduced to rotten fragments and could not be photographed.

The durability of stave pipe when made of good lumber and properly laid is shown by the following extract from a report on the Ogden waterworks to the Ogden council, by N. W. Bethel, city engineer, and the writer as consulting engineer:

> In consideration of the fact that this wooden stave pipe has been in operation for nearly eight years, its present condition is excellent. We have uncovered portions of the pipe throughout its entire length in at least forty different places, and have found without exception the redwood of which the staves are composed to be in a sound condition.

The life of this kind of pipe is prolonged by an earth covering of 2 feet or more. When laid near the surface, with little or no covering of earth, the combined action of earth, air, and sunlight causes decay.

USE.

The most expensive portions of irrigation canals in earth are the ravines. During the last thirty years the common practice in the Rocky Mountain region has been either to go around these ravines with a grade canal or to erect across each a flume on trestles. The first method is often impracticable, and in any event the length of the canal is unnecessarily increased. The latter method proves expensive, not because of the first cost, but on account of the annual expense of maintenance and the frequent renewals. Such flumes usually have to be calked every spring, while both trestles and flume are exposed to landslides, fires, and windstorms, or, escaping these, are inevitably subject to early decay. Stave pipe, on account of its cheapness and the large sizes that can be constructed, is rapidly taking the place of

flumes on trestles. One or more lines of pipe are laid in a trench dug across the bottom of the ravine, and are connected with the grade canal at either end by means of shallow wells or rectangular boxes. The cost of an inverted pipe siphon is frequently less, and it may last two or three times longer than a flume on trestles.

The old-fashioned way of conveying water to gristmills, sawmills, or manufacturing plants was to make use of a short, open raceway from the stream to the mill and utilize the low head thus created. Wooden pipe, on account of its general adaptability, is now being extensively used for this purpose. It permits the conveyance of an equal amount of water a much longer distance, increasing the fall and consequently the power produced.

This kind of pipe is also frequently used to convey water to domestic distributing reservoirs, from which the water is conducted through iron pressure pipes laid in the streets of the city.

COST.

On account of the fluctuation in the prices of lumber and steel, as well as the variation in freight rates and many other causes, it is impossible to state the cost of stave pipe. In the Rocky Mountain States the price of redwood joists dressed into staves is about $40 per 1,000 feet B. M. A pipe 20 inches in diameter requires about 13 feet of lumber to each lineal foot of pipe; a 24-inch pipe requires 16 feet of lumber, a 30-inch pipe requires 18 feet, a 36-inch pipe requires 22 feet, and so on in about that ratio. The cost of the iron and steel per lineal foot of pipe depends on the pressure of water within the pipe. When a stave pipe is laid near grade the bands may be spaced 12 or more inches apart. If, on the other hand, it is laid across the bottom of a ravine 100 feet below grade, the bands should be about 3 inches apart.

The figures given below may convey some idea of the present comparative cost of stave pipe, cast-iron pipe, and riveted-steel pipe in the city of Salt Lake. The following bids were received March 24, 1900, for furnishing all of the materials and laying the City Creek pipe line:

4,390 lineal feet of 30-inch stave pipe, $2.95 and $3.10; or
4,390 lineal feet of 30-inch cast-iron pipe, $10.20 and $10.85; or
4,390 lineal feet of 30-inch riveted-steel pipe, $8.65 and $9.15.
4,390 lineal feet of 24-inch stave pipe, $2.60 and $2.55;
5,920 lineal feet of 24-inch cast-iron pipe, $7.45 and $8.15; and
5,920 lineal feet of 24-inch riveted-steel pipe, $5.75 and $6.05.

RIVETED-STEEL PIPE.

Riveted-steel pipe is extensively used in southern California to convey water for orchard irrigation, but its use in the Rocky Mountain States has been confined to power plants and domestic water supplies.

It is well adapted to high heads, and irrigators who desire to conduct water across deep gulches would do well to write to the makers of this kind of pipe for prices. The principal manufacturers of riveted-steel pipe on the Pacific coast are the Union Iron Works and Francis Smith & Company, of San Francisco, and the Lacy Manufacturing Company and J. D. Hooker, of Los Angeles. The writer has also obtained prices at various times from the Carroll-Porter Boiler and Tank Company, of Pittsburg, Pennsylvania.

Riveted-steel pipe ranges in size from 3 inches to 72 inches internal diameter. The smaller sizes are made into lengths of 25 feet or more at the shops and are thoroughly coated with asphaltum. The larger sizes are usually made in temporary shops erected near the site where the pipe is to be laid.

The following extracts from general specifications furnished by the secretary of the Carroll-Porter Boiler and Tank Company will give the reader some idea of the manner of making this kind of pipe:

Requirements for steel plates.—The steel shall be of the class termed "soft" and shall be made by the basic or acid open hearth process. If by the basic open hearth process it shall contain not more than .04 of phosphorus and .04 of sulphur. If acid it shall contain not over .08 of phosphorus and .05 sulphur.

The amount of manganese and carbon to be in such quantities as shall produce the best results.

Chemical analysis of each "heat" or "melt" shall be made by the Engineer or his representative, and properly certified copies of the final analysis of the material shall be furnished to the Engineer as the work progresses.

Physical tests to determine the tensile strength, elastic limit, elongation, softness, and ductility of the material of each "heat" or "melt" shall also be made by experienced Inspectors whose services may be satisfactory to the Engineer. For the purpose of identification the "heats" or "melts" shall be numbered consecutively, and the corresponding number stamped upon each plate or sheet produced therefrom.. The test specimens shall in all cases be taken from the shearings of such plates produced from each "heat" or "melt" as is thought necessary, and such sheets or plates to be selected at random by said Inspector or Engineer, and properly certified copies of the record of such test shall be furnished the Engineer as the work progresses.

Tensile test specimens to be eight (8) inches long and one and one half (1½) inches wide between measuring points. Tensile strength is to be between the limits of fifty two thousand (52,000) and sixty two thousand (62,000) pounds per square inch. Elastic limit to be not less than one half the tensile strength required. For plates three eighths (⅜) inch thick or more, the elongation to be not less than twenty-five (25) per cent. For plates thinner than three eighths (⅜) inch, elongation to be not less than twenty-two and one half (22½) per cent.

Bending test specimens cut lengthwise or crosswise from the sheet to be six (6) inches long and one (1) inch wide, to be bent one hundred and eighty (180) degrees upon itself when cold, and hammered down flat without sign of fracture on the outside of the bent portion.

The plates must also admit of cold hammering or scarfing to a fine edge at the laps without cracking, and the test pieces must furthermore withstand such quenching, forging, and other tests as may suffice to exhibit fully the temper, soundness and fitness for use of the material. The failure of said test specimens, when taken at ran-

dom as aforesaid from the finished product of any "heat" or "melt," to conform to the above requirements will be sufficient cause for the rejection of the entire product of such "heats" or "melts."

The plates shall be free from lamination and surface defects and be fully up to the required gauge for thickness on the edges. Any plate whose thickness at any point may be found less than ninety-five (95) per cent of the required thickness shall be rejected without appeal; furthermore, at least ninety (90) per cent of the plates must be of the full required thickness at all points. Said plates shall be rolled as flat and sheared as accurately as good mill practice will permit, but in no case shall they be scant of the prescribed or intended dimensions, and must be in all respects of a good merchantable condition. The Engineer shall have the right at all times to inspect the manufacture and testing of any and all plates and shall have, if so required, one fourth ($\frac{1}{4}$) of the number of test pieces to be prepared as above, to test or to have tested under his own supervision.

It is further understood and agreed that any plate that shows any defects during the process of punching, bending, and riveting for manufacturing into pipes shall be rejected, notwithstanding that the same may previously have been satisfactorily tested.

Requirements for steel rivets.—Rivet steel shall be soft and have a tensile strength between the limits of fifty thousand (50,000) and fifty eight thousand (58,000) pounds per square inch, with an elastic limit of not less than thirty thousand (30,000) pounds per square inch, and with an elongation of not less than twenty-eight (28) per cent, in a test bar eight inches long between measuring points and full diameter of rivet, and with a reduction of cross sectional area at the point of fracture of not less than fifty (50) per cent. The material shall also be of such quality as will stand bending double and flat before and after heating to a light yellow heat, and quenching in cold water, without sign of fracture on the convex surface of the bend. All steel rivets not conforming to the above requirements will be rejected.

Manufacture of pipe.—The sheets or plates must be of such dimensions as to admit of being rolled into true cylinders not less than five (5) feet in length and of the required internal diameter with ample allowance for the necessary overlap at the single longitudinal seam of each such cylinder. The pipe shall be made telescopic and the plates when formed shall be a cylinder whose external diameter at one end shall be equal to its internal diameter at the other end and forming a tight fit with each other before any protective coating is applied to the material.

The edges of each plate must be properly beveled for calking all around; and at the end of each course, where the lap of the longitudinal seam occurs, the plate must be reduced by cold hammering or planing, or both, to a fine edge, through which one of the rivets of the round seam must be driven to insure tightness. In addition to this rivet, still another rivet must be driven through the three thicknesses of plate at such joints. Each plate must be rolled to a perfect cylinder of the required diameter.

All rivet holes must be spaced with precision. In punching said holes the best and sharpest dies and punches are to be used. The holes must all coincide to within one thirty-second ($\frac{1}{32}$) of an inch; otherwise they are to be enlarged by a sharp reamer or drill. No drift pins shall be used in forcing rivet holes to coincide at any seam or lap; and all plates in which the said holes cannot be made to receive a rivet of the specified diameter, with such slight drifting or reaming as will not, in the opinion of the Inspector, reduce the strength of the plates, will be rejected.

All riveting in the shop must be done with hot steel rivets and by steam, compressed air, or hydraulic machinery exerting a slow pressure and retaining it sufficiently to perfectly form the rivet heads before the metal loses its red color. All rivets when driven must fit the holes completely. Care must in all cases be taken to have the rivets of the right heat to produce the best and most solid work, and any

rivet that may at any time or place be found defective or appear to have been overheated, shall be cut out and replaced at the Contractor's expense by one which is sound in all respects. All rivet heads must be perfect in form and truly concentric with the shank or shaft.

All circular seams may be single riveted but all longitudinal seams on the straight pipe shall be double riveted. The proportions to be used in laying out and making the riveted joints or seams shall in general be as follows, all figures being in inches.

* * * * * * *

The dimensions given in the foregoing table are to be regarded as approximate, and subject to modification before the material for the pipe is definitely ordered by the Contractor; and any modification proposed by said Contractor shall be subject in detail to the approval of the Engineer before being carried into effect.

The pipe is to be manufactured in lengths of four or more courses each, and so laid that all longitudinal seams will at all times be on top and within a foot of the center line of the pipe.

Where angles or curves occur in either the alignment or the grade of the conduit, the plates must be cut and punched to the required lines for forming a small oblique angle at the round seams of as many courses as may be needed to produce the given total deflection or curvature in each locality, and the courses must be put together with the straight seams alternating as aforesaid. Where greater angles and special pieces are necessary they shall be made as ordered by the Engineer.

All rivet seams and joints of every description shall be thoroughly calked both on the inside and outside in the best and most workmanlike manner for first class boiler work, while for the necessary distance from all laps the seams shall be both chipped and calked.

Testing.—During its manufacture, sample lengths of pipe to be selected by the Inspector as frequently as he may deem necessary, shall be tested after coating, under a water pressure equal to one and one half (1½) times the maximum hydrostatic head for which it is designed, and shall be tight under that pressure. All such tests will be made at the Contractor's expense, and he shall furnish all the necessary appliances and labor for their performance to the Inspector's satisfaction.

Coating.—When the pipes are finished, they are to be thoroughly coated by dipping in a bath of refined asphaltum, as follows: Each length of pipe must be thoroughly clean inside and outside, free from dust, earth or sand, and without rust upon any part of it. Any rust that may have formed must be removed by brushing and scrubbing with a wire brush and diluted acid, followed by mopping or brushing with milk of lime, or a saturated solution of soda, to remove the acid, until all the rust and acid shall have been removed. All alkali used shall be washed off and the surface dried.

The pipes are then to be heated, after which they are to be dipped in the bath of coating material. The coating shall consist of the best quality of California or Trinidad refined asphalt, such as is used for pipe coating purposes. The coating must be durable, smooth, glossy, hard, tough, perfectly water-proof, not affected by any salts or acids found in the soil, strongly adhesive to the metal under all circumstances, and with no tendency to become soft enough to flow when exposed to the sun in summer, or to become so brittle as to scale off in winter.

After having been prepared as aforesaid, the coating composition shall be placed in a vertical tank and carefully heated to a temperature of about three hundred (300) degrees F., or such other temperature as may hereafter be found best adapted for the purpose, and kept at this temperature during the whole time of dipping.

Fresh materials must be added to the aforesaid bath from time to time in the right proportion to keep the mixture of the proper consistency. Such proportions may also be varied according to the season of the year, as will be directed by the Engineer or found necessary to produce a coating of the required quality.

CAST-IRON PIPE.

HISTORY.

According to Mr. Jesse Garrett,[1] the first cast-iron pipe was used for the water service of Versailles, France, in 1685. The first iron pipe was laid in London in 1746, and in 1835 the various water systems of that metropolis included more than 1,000 miles of that kind of pipe.

During the early part of this century James Watt laid cast-iron pipes with flexible joints across the River Clyde, to convey water to Glasgow. These pipes had ball and socket joints, were 9 feet long, and from 15 inches to 36 inches in diameter.

The first cast-iron pipes used by the city of Philadelphia were molded by Mark Adams from designs prepared by Frederic Graff, sr., the engineer of the Philadelphia waterworks. These pipes were cast direct from melted native bog ore. The pipe molds were placed in a nearly horizontal position and sufficiently low to tap the flow from the furnace, the excess of each pouring being run into pigs.

Fanning states[2] that the earliest pipes had joints with a packing ring of leather. These were 2½ feet in length. Then followed somewhat longer pipes, with screw joints, which, however, were not satisfactory, the rigid joints preventing free contraction in winter and expansion in summer. For screw joints there were soon substituted slightly tapering cylindrical socket joints, in which both hub and spigot ends were accurately dressed on a lathe. In forming the joint, a heavy lubricant, like tallow, was first spread over the dressed surface, after which each spigot end was driven into its socket. This kind of joint is still used for water and gas pipes in parts of Great Britain and Canada.

The bell and spigot joint so common at the present time was first adapted to wood instead of lead packing. Dry pine, dressed in the form of a stave to suit the diameter of the pipe to be calked and about 3 inches in width, was cut into lengths a trifle longer than the depth of the socket. From four to eight of these pine wedges, which tapered slightly, were entered at the top, bottom, and middle of the joint and driven home together. The remaining annular spaces were filled by other wedges somewhat wider than the spaces.[3]

MANUFACTURE.[4]

The first process in the manufacture of cast-iron pipe consists in making the core, which is formed on a cylindrical iron spindle. The

[1] The Early History of Cast-Iron Pipe, by Jesse Garrett; Read before the New England Water Works Association in 1896.

[2] Hydraulic Engineering and Manual for Water Supply Engineers, p. 458.

[3] For description of pine packing in cast-iron pipes see paper by F. A. Creighton in Trans. Can. Soc. Civ. Engrs., Vol. VIII, Pt. I, p. 145.

[4] The writer is indebted to Mr. W. F. McCue, of the Colorado Fuel and Iron Company, for much of the information contained in this description.

spindle is first wrapped with machine-made hay rope, and around the rope tempered clay is firmly packed. After this coating is dried a second coat is put on. Then the spindle is revolved in its bearings and a third coat, consisting of blacking made from coal dust, is added. After this last coat has been applied the core is trimmed to exact dimensions and is then placed in a drying oven.

The next step in the process is to center a cylindrical iron pattern inside of the flask in which the mold is formed. The outside diameter of this pattern is the inside diameter of the pipe to be made. Sand and a small portion of clay slightly tempered are firmly rammed around the pattern, so as to form the mold. The flask is then ready for the drying pan. When the several parts are sufficiently dried they are taken from the oven and placed in the casting pit. The core is placed in the molds and is accurately centered in the bevel at the bottom and to the bead ring at the top in order to insure the desired dimensions of pipe. The molten metal is taken from the cupola in a ladle swung from a crane and is poured into the molds around the core. The intense heat from the metal causes the hay rope on the spindle to burn, leaving the spindle loose and capable of being easily withdrawn. As soon as the metal has become sufficiently cooled the pipe is lifted from the flask, taken outside of the building and placed upon skids, where it is thoroughly cleaned and prepared for the coating. After being cleaned it is placed on a car, run into an oven, and heated to a temperature of 300° F. It is then removed from the oven and immersed in a bath of coal pitch and varnish, which is maintained at a temperature of about 210°. After the coating has dried the pipe is weighed and the weight is marked with white-lead paint on the inside of the bell end. It is then subjected to a hydrostatic test, the pressure varying from 250 to 300 pounds per square inch, according to the thickness of the shell and the size of the pipe, and while under this pressure it is subjected to an additional test, consisting of a series of blows from a 3-pound hammer attached to a handle 16 inches long, the blows being applied by hand at various points throughout the length of the pipe.

DIMENSIONS AND WEIGHTS.

A few years ago a committee was appointed by the American Water Works Association to draft suitable specifications for standard cast-iron water pipe. Mr. S. B. Russell, who was chairman of the committee, recommended[1] three weights for each size of pipe, to be designated A, B, and C. The weight of class B was determined from the average weight of the pipe used in a large number of American cities

[1] Report of proceedings of the annual meeting of the American Water Works Association (1890), p. 23.

and towns; class A was 10 per cent lighter than class B, and class C was 10 per cent heavier than class B. The following table embodies the recommendations of the committee regarding the outside diameter, the average thickness, and the weight of each 1,000 feet of each size and class. To determine the weight per lineal foot, including bell ends and spigots, the weights given should be divided by 1,000.

Dimensions and weights of cast-iron water pipe adopted by American Water Works Association in 1890.

Size (internal diameter).	Class.	Outside diameter.	Average thickness of barrel.	Weight per 1,000 feet.[a]
		Inches.	*Inches.*	*Pounds.*
3-inch	A			14,000
	B	3.856	0.428	15,580
	C			17,170
4-inch	A			19,170
	B	4.904	0.452	21,330
	C			23,500
6-inch	A			31,330
	B	7.000	0.500	34,830
	C			38,330
8-inch	A			44,750
	B	9.094	0.547	49,750
	C			54,750
10-inch	A			60,570
	B	11.190	0.595	67,330
	C			74,080
12-inch	A			78,250
	B	13.286	0.643	86,920
	C			95,580
16-inch	A			119,170
	B	17.476	0.738	132,420
	C			145,580
20-inch	A			165,330
	B	21.666	0.833	183,750
	C			202,080
24-inch	A			220,750
	B	25.856	0.928	245,250
	C			269,750
30-inch	A			316,250
	B	32.142	1.071	351,420
	C			386,580
36-inch	A			428,750
	B	38.428	1.214	476,420
	C			524,080
48-inch	A			710,670
	B	51.000	1.500	789,670
	C			868,670

[a] Including bell ends and spigots.

The following table gives the dimensions and weights per lineal foot of the various sizes of cast-iron water pipe made by the Colorado Fuel and Iron Company, of Pueblo, Colorado. The thickness of each shell is calculated to withstand a hydrostatic pressure of 125 pounds per square inch.

Dimensions and weights of cast-iron water pipe made by the Colorado Fuel and Iron Company.

Size (internal diameter).	Weight per foot.	Inside diameter of bell at face.	Outside diameter of bell at face.	Outside diameter at spigot.	Thickness of shell.
	Pounds.	*Inches.*	*Inches.*	*Inches.*	*Inch.*
3-inch	15	4½	7	4⅜	13/32
4-inch	22	5½	8¼	5⅝	7/16
6-inch	32	7⅝	10⅞	7½	½
8-inch	42	9⅞	13⅜	9¾	17/32
10-inch	62	11⅞	15⅝	11¾	19/32
12-inch	75	13¾	17¾	13⅞	⅝
16-inch	125	18	22½	17⅞	¾
20-inch	185	22⅛	26⅞	22	27/32

TESTING AND INSPECTING.

The defects common to cast-iron pipe are chiefly due either to the use of poor material or to bad workmanship. The essential elements in cast iron are metallic iron and a low percentage of carbon. Its quality depends to a large extent on the amount of impurities it contains. The presence of sulphur produces a hard, weak, and brittle metal. W. F. Keep states[1] that good foundry iron should contain not more than 0.15 per cent of sulphur nor more than 0.55 per cent of phosphorus. From 2 to 5 per cent of phosphorus produces a brittle metal. The same writer adds that the addition of a small amount of silicon eliminates blowholes and produces sound castings, but that an excess of silicon increases the shrinkage and hardness.

The quality of the material can best be determined by testing samples. Samples for the beam or tranverse test are usually 2 inches wide, 1 inch deep, and 24 inches between supports. Such a bar should support a weight of 2,000 pounds at the center and bend at least one-fourth inch before breaking.

The tenacity of the metal is commonly found by testing small specimens having cylindrical centers and enlarged ends. The specimen is first dressed to a known area (1 square inch) at the middle section, and is then placed in the grip blocks and pulled until broken. An average grade of iron suitable for water pipe should withstand a strain of 20,000 pounds per square inch before breaking.

A common error in the manufacture of cast-iron pipe is to remove the pipe from the flasks before it is properly cooled. Unequal cooling has been known to reduce the strength of small test samples 10 per cent. Besides, these cooling strains are very difficult to detect. It not infrequently happens that a pipe which has been tested in the proving press to 300 pounds per square inch fails when laid in the ground and subjected to bending stresses and water ram. J. Nelson Tubbs, formerly chief engineer of the Rochester waterworks, made use of a steel pick weighing about 3 pounds to test pipes for internal

[1] See The Materials of Construction, by J. B. Johnson, p. 97.

cracks caused by unequal cooling, honeycombed shells, and other defects.

The pressure test consists in placing each length of straight pipe in a proving press and increasing the water pressure to 300 pounds per square inch. While the pipe is under pressure it is struck at different places throughout its entire length by a 3-pound hammer attached to a handle 16 inches long.

Another defect, caused by careless molding, is unequal thickness in the shell of the pipe. Heavy-sided pipes can usually be detected by placing them on horizontal skids. If any doubt exists as to the uniform thickness of the shell, the pipe should be set aside and carefully calipered. A difference of 10 per cent in the thickness of the pipe at different points should warrant its rejection. The real test, however, comes after the pipe has been laid and connected. The whole system should then be subjected to a pressure somewhat in excess of any it will likely be called upon to withstand, and this pressure should be increased rapidly and shut off quickly, in order that the pipe may be tested for the extra strain due to pressure suddenly raised and lowered.

LAYING.

It is best to lay the outlet pipes of storage reservoirs in the natural soil, on stable, uniform beds, with wing walls of cement near the upper end and near the center, to prevent the passage of water along the outside of the pipe.

If it is necessary to lay the outlet pipe in made ground, a parabolic curve with the inner end at the vertex is perhaps the best, as the arched form increases the strength of the pipe and the subsidence of the embankment will tend to tighten the joints.

Pipe lines should be accurately located, either by transit lines or by measurements from permanent objects. The location of all fittings, valves, hydrants, etc., should also be shown on the maps. The depth of the trench depends, as a rule, on the amount of frost. In Leadville, Colorado, water pipes have frozen at a depth of 6 feet, while in California a covering of 18 inches is ample.

The pipe should be inspected while it is being removed from the cars, the weight of each length being noted, and then distributed along the trench line, with the bell ends forward. After the bottom of the trench has been reduced to a uniform grade, a hole should be dug at the place for each joint, for the reception of the bell end, and the pipe should be lowered into the trench, by means of derricks or chain falls, and calked. The practice of leading several lengths of pipe on the bank and lowering two or three lengths at a time into the trench is not to be encouraged, because the joints are often sprung in lowering and placing. In leading, a part of the vacant space in each bell surrounding the spigot is first filled with yarn or oakum, inserted by the use of a yarning tool; the remaining space is filled with lead and thoroughly

calked. Narrow-edged calking tools are first used, to compact the metal at the inner edge of the lead space; these are followed by tools that nearly fill the joint.

In back filling the trench the earth should be carefully tamped beneath and at the sides of the pipe until it is about two-thirds covered. Earth bridges can then be made, to prevent flotation, and the trench partially filled with water, after which the remainder of the earth can be filled in without tamping.

In American practice the usual form of joint consists of a spigot inserted into a bell or socket and calked with yarn and lead. In Great Britain, however, and occasionally on the Continent, the turned and bored joint is still used. In this form of joint the socket is bored to a depth of 4 or more inches, the spigot end is turned to fit the socket, and after painting, the spigot end is inserted and driven into the drilled socket. Both of the dressed surfaces are slightly conical.

Owing to the hemp packing and the softness of the lead, a lead joint is much less rigid than a turned and bored joint, and conforms more readily to uneven grade lines and to expansion and contraction produced by unequal temperature. On the other hand, lead joints are more likely to leak after a sudden change in the temperature. Leaky joints are common in the spring of the year when the frost is leaving the ground. Both lead and cast iron contract in the winter and the joints remain tight, but with the advent of spring both expand, but in unequal ratios—cast iron expands nearly three times as much as lead—and leaks frequently result.

The following table gives the approximate cost of the various items in laying cast-iron water pipe. The amount of lead required for a joint depends upon the size and shape of the bell and spigot, which vary at the different manufactories; also upon the degree of water tightness desired. In the sixth column of the table is given the cost of trenching and back filling as computed by Mr. James B. Hopkins, who was for years the writer's foreman.

Table showing cost, per lineal foot, of laying cast-iron water pipe.

Size (internal diameter).	Weight of filling, per joint.		Cost of lead and oakum, per foot of pipe.	Cost of laying, per foot.	Cost of trenching and back filling, per foot.	Total cost, per lineal foot.
	Lead.	Oakum.				
	Pounds.	*Ounces.*				
4-inch	7	3	$0.04	$0.02	$0.08	$0.14
6-inch	10	5	0.06	0.02	0.08	0.16
8-inch	13	7	0.07	0.03	0.09	0.19
10-inch	15	9	0.09	0.03	0.09	0.21
12-inch	18	11	0.10	0.04	0.10	0.24
14-inch	22	13	0.12	0.04	0.10	0.26
16-inch	26	15	0.14	0.045	0.11	0.29
18-inch	32	16	0.17	0.05	0.12	0.34
20-inch	38	18	0.20	0.06	0.12	0.38
24-inch	42	20	0.23	0.08	0.14	0.45
30-inch	60	24	0.32	0.16	0.16	0.64
36-inch	80	30	0.43	0.18	0.20	0.81

DURABILITY.

There is, properly speaking, no substitute for cast iron as a material for water mains. Cast-iron pipe has stood the test of a century, and is now universally regarded as superior to all other pipes for the conveyance of water. Hydraulic engineers have recommended it as safe and reliable, and private and municipal corporations have purchased and laid millions of tons in the various waterworks systems of the country. It is true that substitutes have been introduced in small cities and towns of the West, where the cost of transportation is high, and while some of these substitutes have given satisfaction the majority have proved partial failures at least. On permanent streets, beneath reservoir embankments, and wherever durability is an essential element, cast-iron pipe should be used. For waterworks conduits which may soon be replaced by larger ones, and for irrigation works, there are other kinds of piping that may be preferred.

COST.

In the Rocky Mountain States the price of cast-iron pipe has decreased from about $42 a ton in 1888 to about $30 a ton in 1898.

In the following table three different weights—light, medium, and heavy—are included. The light-weight pipe should safely withstand a static pressure of 120 pounds per square inch as well as ordinary water ram.

Table showing cost, per lineal foot, of cast-iron pipe at $30 a ton.

Size (internal diameter).	Light-weight pipe.		Medium-weight pipe.		Heavy-weight pipe.	
	Weight per foot.	Price per foot.	Weight per foot.	Price per foot.	Weight per foot.	Price per foot.
	Pounds.		*Pounds.*		*Pounds.*	
3-inch...	14	$0.21	15	$0.23	17	$0.26
4-inch...	19	0.29	21	0.32	23	0.35
6-inch...	31	0.47	35	0.53	38	0.57
8-inch...	45	0.68	50	0.75	55	0.83
10-inch...	61	0.93	67	1.01	74	1.11
12-inch...	78	1.17	87	1.31	96	1.44
16-inch...	119	1.69	132	1.98	146	2.19
20-inch...	165	2.48	184	2.76	202	3.03
24-inch...	221	3.32	245	3.68	270	4.05
30-inch...	316	4.74	351	5.27	386	5.79
36-inch...	429	6.44	476	7.14	524	7.86
48-inch...	711	10.67	790	11.85	869	13.04

The cost of laying the pipe has been given in the table on page 80.

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DEPARTMENT OF THE INTERIOR

WATER-SUPPLY

AND

IRRIGATION PAPERS

OF THE

UNITED STATES GEOLOGICAL SURVEY

No. 44

WASHINGTON
GOVERNMENT PRINTING OFFICE
1901

UNITED STATES GEOLOGICAL SURVEY

CHARLES D. WALCOTT, DIRECTOR

PROFILES OF RIVERS

IN THE

UNITED STATES

BY

HENRY GANNETT

WASHINGTON
GOVERNMENT PRINTING OFFICE
1901

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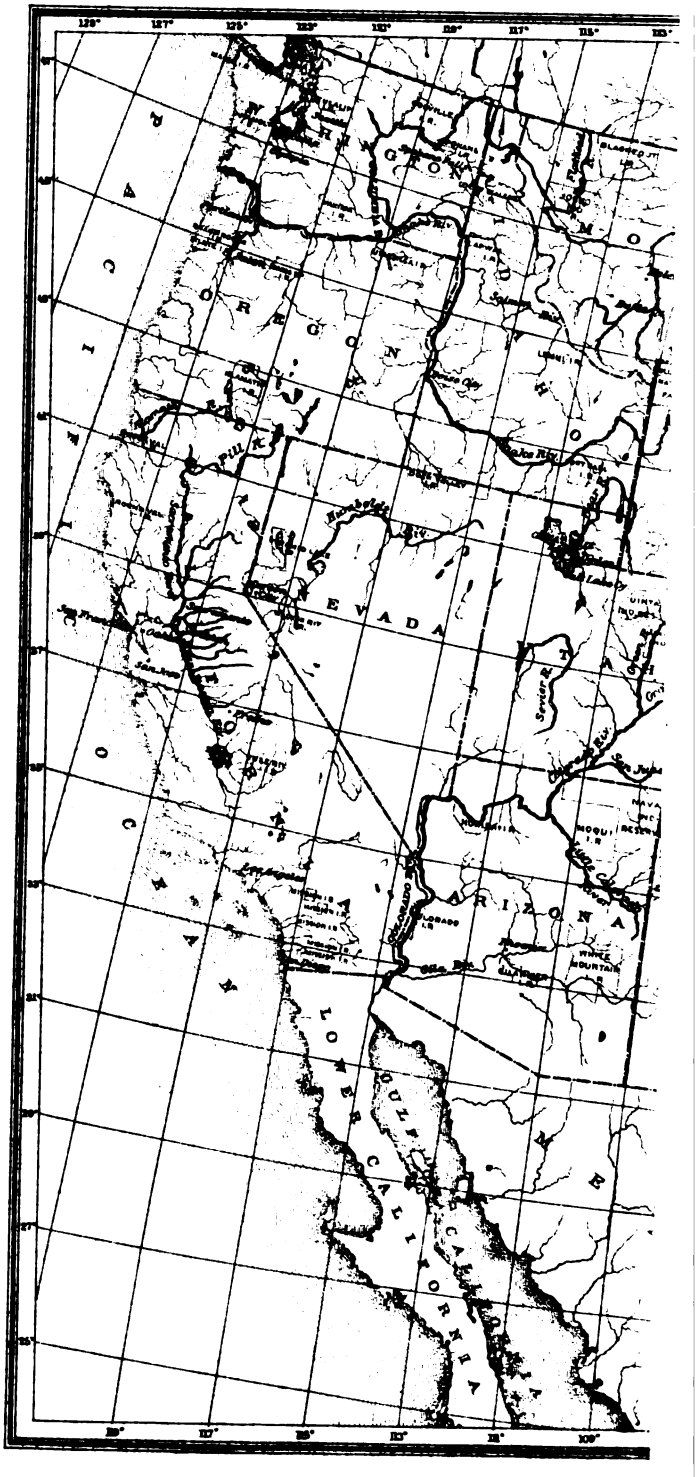
U.S. GEOLOGICAL SURVEY.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GULF OF CALIFORNIA
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Snake R.
Sevier R.
San Juan
Colorado R.
Gila R.
Los Angeles
San Francisco
Salt Lake City
Ogden
Pocatello
Boise City
Walla Walla
Spokane
Yakima
Tacoma
Seattle
Portland
Eugene
Sacramento
Stockton
Fresno
Bakersfield
Los Angeles
San Diego
Ensenada
Yuma
Prescott
Flagstaff
Phoenix
Tucson
Winslow
Needles
Kingman
Ft. Mohave
Ft. Yuma
Ft. Grant
Ft. Bowie
Ft. Apache
Ft. Wingate
Ft. Defiance
Ft. Lewis
Ft. Hall
Ft. Boise
Ft. Klamath
Ft. Vancouver
Ft. Simcoe
Ft. Spokane
Ft. Colville
Ft. Walla Walla
Ft. Yakima
Ft. Lapwai
Ft. Bidwell
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Ft. Mojave
Ft. Yuma
Ft. Grant
Ft. Bowie
Ft. Apache

PROFILES OF RIVERS IN THE UNITED STATES.

By Henry Gannett.

GENERAL DISCUSSION.

The profiles here represented are derived from various sources and differ from one another greatly in accuracy. Many of them are drawn from the annual reports of the Chief of Engineers, U. S. A., under which are included the reports of the Mississippi and Missouri River commissions. The heights thus obtained are those of the level of water in the rivers at certain stages, and may be regarded as of great accuracy. Others have been obtained from railroad profiles, being the level of the rivers at points where the railroads touch or cross them. Still others have been taken from the atlas sheets of the United States Geological Survey and from other maps. In most such cases the points at which the contours cross the rivers upon the maps have been taken. These again differ in point of accuracy with the means adopted for the location of the contours. Where the spirit level was used in locating contours it may be assumed that the determinations are fairly good, but where the barometer was used the probable error may be of considerable magnitude. Such elevations are, however, mainly in the mountainous parts of the country, where the fall of the streams is great, and where, therefore, errors of considerable magnitude may be tolerated, as affecting but little the form of the profile.

The rivers whose profiles are presented in the following pages are indicated upon the map which forms Pl. I. The profiles are given in figures, showing the distance between points, the height at each point, and the average fall per mile between points. They are also represented graphically upon Pls. II to XI, inclusive. All these profiles are represented upon the same horizontal and vertical scales, the former being 100 miles to an inch and the latter 2,000 feet to an inch. This relation between the scales, which scarcely suffices to show any slope in the Lower Mississippi, gives the appearance to many other streams of exceeding steepness, as in the case of those flowing out of the Sierra Nevada. Still it was judged best, after much consideration, to use uniform scales throughout, in order that comparisons between different rivers might be made directly.

ST. CROIX RIVER.

The drainage basin of this river is in eastern Maine, a portion of it projecting over into New Brunswick. Its area is 1,674 square miles. Its surface is undulating and hilly, but not mountainous, and is, in the main, densely forested.

At a distance of 20 miles from its mouth the river forks, the main branch, known as the Kennebasis, coming down from the north. It is of this branch that the profile is given. The other branch, known as the Chiputneticook, comes from the west.

To illustrate the lacustrine character of this stream it may be stated that there are within its drainage basin, and tributary to it, no fewer than 27 lakes, ranging in size from three-quarters of a square mile up to 27 square miles, and with a total area of 134 square miles.

Upon the river and its northern tributary there are many falls and rapids, only a few of which have been utilized.

The profile shows great and recent disturbances, changing it materially from the normal profile which it must have presented at the opening of the Glacial epoch.

The figures are from Wells's Water Power of Maine.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Upper bridge, Milltown, mouth	0	0	.....
Baring bridge	5¼	86	.....
Foot of Sprague falls	10¼	93	9
Head of Sprague falls	10¾	118	50
Head of Enoch rips	11¼	128	.....
Pitch of Lower Grand falls	19¼	145	3
Head of Upper Grand falls	19¾	165	40
Foot of Grand Chiputneticook falls	22	169	2
Head of Grand Chiputneticook falls	22¾	190	28
Foot of Canoose rips	30	200	1
Head of Canoose rips	30½	211	22
Foot of Haycock rips	33¼	212	.....
Head of Haycock rips	33¾	218	12
Foot of Meeting-house rips	34¾	218	.....
Head of Meeting-house rips	35¾	226	8
Foot of Rocky rips	36½	227	.....
Head of Rocky rips	39¾	252	8
Foot of Mile rips	50	353	.....
Head of Mile rips	51	358	5
Foot of Kill-me-quick rips	54	372	5
Head of Kill-me-quick rips	54½	382	20
Head of Chiputneticook Lake	74½	382	.....
Stream into Mud Lake	76¼	426	22
Head of Mud Lake	80½	426	.....

PENOBSCOT RIVER.

This, the largest river in the State, has a drainage area of 8,934 square miles, its greatest length being 160 miles, and its greatest breadth 115 miles. Its surface is hilly or undulating, in the main forest-clad and full of lakes; indeed, within its basin 185 lakes and ponds have been counted, having a total area of 395 square miles.

The course of the river is crooked; indeed, in its upper part it has apparently no definite direction, flowing through lakes and swamps. It meets tide at Vinal Mills, 39 miles above its mouth. Its profile is equally irregular, consisting of level reaches, rapids, and falls.

The profile is from the census report on Water Power, 1880.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	0	.....
Bangor	27	0	.....
Head of falls, Vinal Mills	39	92	7.7
Mouth of Mattawamkeag River	84	190	2.2
Chesuncook Lake	120	900	19.7
Penobscot Lake	200	1,509	7.6

KENNEBEC RIVER.

This river has its source in Moosehead Lake, from which it flows in a generally southward direction to its mouth below Bath. It meets tide at Augusta, 26 miles above Merrymeeting Bay, and about 40 miles above its mouth. Its drainage area, excluding Androscoggin River, is 6,400 square miles, most of which, and especially the northern part, is densely forested. It is in a lacustrine region containing hundreds of lakes and ponds, most of which are small in area. The profile is from the census report on Water Power, 1880.

Locality.	Distance from Merrymeeting Bay.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Merrymeeting Bay	0	0	.....
Augusta, head of tide	26	0	.....
Head of Kendall Mills rip	48	91	4.1
Norridgewock	66	138	2.6
Dam at Madison bridge	79	236	7.5
Head of Caratunk falls	90	316	7.3
Moosehead Lake	138	1,023	14.7

ANDROSCOGGIN RIVER.

This river heads in the White Mountains, in New Hampshire, and flows in a generally eastward and southeastward direction, joining the Kennebec in Merrymeeting Bay. Its drainage basin presents the same characteristics as those of the Penobscot and Kennebec, varied by the fact that its sources are in a mountain region and at a considerable elevation. It has the same forest-clad, lacustrine character. The river contains numerous falls, many of which, especially at Lewiston, have been greatly utilized. The total area drained measures 3,698 square miles, of which three-fourths lie in Maine and the remainder in New Hampshire. The profile is from the census report on Water Power, 1880.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth, Merrymeeting Bay	0	0	.....
Head of Rumford falls	75	600	8.0
Bethel	100	620	0.8
State line	114	690	5.0
Head of Berlin falls	128	1,048	25.6
Head of river proper	160	1,256	6.5
Parmachene Lake	186	1,600	13.2
Magalloway Lake	199	2,225	48.1

SACO RIVER, MAINE AND NEW HAMPSHIRE.

This stream heads in eastern New Hampshire and flows east and southeast to its mouth, near Saco and Biddeford. Its drainage basin, having an area of 1,750 square miles, half of which is in Maine and half in New Hampshire, is quite similar in its characteristics to the basins of the other rivers of these States. Its head is in the southern part of the White Mountains, and it flows thence through the low country in Maine.

The profile is from the census report on Water Power, 1880.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	0	.....
Mouth of Ossipee River	40	266	6.7
Foot of Great falls	45	271	1.0
Head of Great falls	45	343	.....
Conway Center	73	412	2.5
Mouth Ellis River	83	511	9.9
West boundary of Bartlett	92	745	26.0
Head	104	1,880	94.6

MERRIMAC RIVER.

This little stream, which probably turns more spindles than any other stream of the United States, heads in the central part of New Hampshire, and flows in a direction a little east of south into northeastern Massachusetts, which it traverses in a northeast course to its mouth, at Newburyport. Its drainage basin is 4,864 square miles. The upper portion is a forest-clad, lacustrine region, but it soon enters a country which is densely settled and has been largely cleared of forests. Everywhere, however, it abounds in lakes and ponds. Its profile shows the irregularities which are so characteristic of the streams of New England, consisting of a succession of still reaches alternating with falls and rapids.

The profile is from the census report on Water Power, 1880.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	0	.....
Mitchell falls	22	0	.....
Foot of locks at Lawrence	27	10	2.0
Top of Lawrence dam	28	39	29.0
Foot of Hunt falls	37	42	0.3
Head of Hunt falls	38	53	11.0
Top of Pawtucket dam, Lowell	40	87	17.0
State line	49	90	0.3
Mouth of Nashua River	53	93	0.8
Foot of Cromwell falls	57	93	0.0
Foot of Goff falls	64	112	2.7
Head of Goff falls	65	117	5.0
Manchester, below falls	68	126	3.0
Top of Manchester dam	69	178	52.0
Hooksett, below falls	78	181	0.3
Hooksett, top of dam	79	197	16.0
Foot of Garvin falls	83	199	0.5
Head of Garvin falls	83	227	28.0
Foot of Sewell falls	93	229	0.2
Head of Sewall falls	95	248	9.5
Mouth of Contoocook River	96	249	1.0
Franklin, head of river	110	269	1.4

CONTOOCOOK RIVER.

This is a branch of Merrimac River in New Hampshire. It heads in the southwestern part of the State, near the Massachusetts line, and flows northeastward to its connection with the parent stream. It has a rapid fall, supplying many water powers.

The profile is from the census report on Water Power, 1880.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	249	.....
Just above Contoocook	11	365	10.5
Just below Henniker	21	389	2.4
Foot of Long fall	22	433	44.0
Head of Long fall	24	546	56.5
Foot of falls at Hillsboro bridge	27	564	6.0
Head of falls at Hillsboro bridge	27	591	.....
Foot of falls at Bennington	36	606	1.7
Head of falls at Bennington	37	676	70.0
Foot of falls at North Peterboro	44	714	5.4
Head of falls at North Peterboro	44	724	1.5
Foot of falls at Peterboro	46	727	
Head of falls at Peterboro	46	734	35.25
Hillsboro county line	50	875	
Three Ponds, in Rindge	55	1,114	47.8

CONNECTICUT RIVER.

Connecticut River heads near the boundary line between the State of Vermont and Canada in a series of small lakes at an altitude exceeding 2,000 feet, flows thence in a southerly course, forming the boundary between Vermont and New Hampshire, and thence across Massachusetts and Connecticut to its mouth in Long Island Sound. Its total length is nearly 400 miles, and its fall in that distance somewhat more than 3,000 feet, being an average of about 5 feet per mile. It is tidal to Hartford, 50 miles above its mouth. The area of its basin includes 11,269 square miles. The upper part of this is forest clad, but lower down, and especially throughout Massachusetts and Connecticut, the forests have been largely cleared away for settlement. Throughout it is in a lacustrine region. From its head as far as central Massachusetts it has a narrow valley, but in southern Massachusetts and northern Connecticut the valley expands broadly, owing to the fact that it is here composed of Triassic sandstones, which are much softer than the Archean rocks in which its course lay above. Below Holyoke its course is crossed by a dike of trap which has retarded its work of eroding its bed, forming a partial dam, behind which the river becomes very gentle, sluggish, and winding, developing a broad course in its bottom land.

One of its curves is known as the Oxbow, which has been cut off, and has become a crescent-shaped lake, similar to those so common in the valley of the Lower Mississippi.

The profile is from the census report on Water Power, 1880.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	0	.....
Hartford	50	0	.....
Foot of Enfield rapids	60	6	0.6
Top of Enfield dam	66	38	5.3
Top of Holyoke dam	84	98	3.3
Fitchburg R. R. crossing	115	109	0.35
Top of Turners Falls dam	120	173	12.8
Mouth of Ashuelot River	136	206	2.1
Westmoreland	159	219	0.6
Foot of Bellows falls	170	234	1.4
Head of Bellows falls	170	283	.....
Beaver Meadows, Charlestown	181	289	0.5
Windsor	196	304	1.0
White River Junction	209	339	2.7
Ledyard bridge, Hanover	213	375	9.0
Oxford	230	380	0.3
Wells River	255	407	1.1
Foot of McIndoes falls	262	432	3.6
Lower Waterford	273	643	19.2
Head of Fifteen-mile falls	285	830	15.6
North Stratford	312	885	2.0
West Stewartstown	344	1,035	4.7
Connecticut Lake	361	1,618	34.3
Second Lake	369	1,882	15.5
Third Lake	375	2,038	26.0

HOUSATONIC RIVER.

This important stream heads in western Massachusetts, flows southerly down into Connecticut, and then turns to the southeast, emptying into Long Island Sound near Bridgeport. It has a drainage basin of 1,933 square miles. In its upper course it flows through a hilly country, partially cleared, the hills gradually becoming smaller and the proportion of woodland less as the stream is traced southward.

The profile is from the census report on Water Power, 1880.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	0	.....
Mouth of Shepaug River	30	105	3.5
Two miles above Cornwall bridge	64	457	10.2
Falls village	73	622	19.4
Ashley Falls, Mass	82	705	9.2
Pittsfield	123	983	4.3

HUDSON RIVER.

This river heads in Lake Tear of the Clouds, in the heart of the Adirondacks, at a great altitude, and comes down the south slope of that mountain group with a steep descent, flowing through many ponds and over many cataracts and waterfalls in its course. The profile rapidly flattens as the river leaves the mountains, but in the neighborhood of Glens Falls, near its junction with the Mohawk, it makes a series of abrupt drops, due to passing over hard rock beds, which furnish valuable water power. It reaches tide at Troy, 150 miles from its mouth.

Its drainage area, including Mohawk River, is 13,366 square miles.

The profile is from the census report on Water Power, 1880.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	0	.....
Troy	150	5	0.03
Crest of Saratoga dam	180	102	3.2
Fort Edward R. R. bridge	190	118	1.6
Crest of Glens Falls feeder dam	197	284	23.7
Mouth of Secondaga River	216	536	13.3
Mouth of Stony Creek	222	571	5.8
Mouth of Schroon River	228	594	3.8
The Glen	236	720	15.7
Mouth of Mill Creek	240	817	24.2
Mouth of North Creek	248	981	20.5
North of River village	253	1,041	12.0
Mouth of Boreas River	257	1,134	23.3
Mouth of Indian River	265	1,403	33.6
Mouth of Cedar River	266	1,454	51.0
Lake Tear of the Clouds	300	4,322	84.4

MOHAWK RIVER.

This, the main western branch of Hudson River, flows eastward through a broad depression between the Adirondacks and the Helderberg Plateau. The irregularities of its profile are due mainly to its flowing over the edges of hard rock beds. Its principal fall is that at Cohoes, near its mouth.

The profile is from the census report on Water Power, 1880.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	12	.....
Lower aqueduct (fall nearly all at Cohoes)	4	162	37.5
Schenectady	19	214	3.5
Mouth of Schoharie Creek	42	270	2.4
Three miles east of Utica	95	393	2.3
Four miles east of Rome	112	418	1.5
Rome, above feeder dam	115	431	4.3

PASSAIC RIVER.

[From the census report on Water Power, 1880.]

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	0	.....
Passaic	29	0	.....
Dundee dam, crest	30	22	22.0
Paterson, below falls	41	39	1.5
Paterson dam, crest	42	115	76.0
Little falls, dam	47	162	9.4
Lower Chatham bridge	69	168	0.3
Near Madisonville	86	240	4.2

DELAWARE RIVER.

This river heads in the western slopes of the Catskill Mountains and in the plateaus north of them, flows in a course alternating between southwest and southeast, but generally southeast, to its mouth, in Delaware Bay, reaching tide at Trenton. Throughout most of its course it forms the boundary line between Pennsylvania, on the west, and New York or New Jersey, on the east. The length of the stream,

following its windings from its source to Trenton, is about 280 miles, and its average fall about 6.7 feet per mile. Its drainage area, including all its branches, is 12,012 square miles.

The first part of its course is down the declivities of the Catskill Mountains. Shortly thereafter it enters the faulted and folded region of the Appalachian Valley, flowing alternately with the ridges and across them. At Delaware Water Gap it passes the Kittatinny Range. At Trenton, where it crosses the Fall Line and meets tide, there is a fall of nearly 8 feet, which furnishes a valuable water power.

The profile is mainly from the census report on Water Power, 1880.

Locality.	Distance from Trenton.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Trenton, below falls	0	0	.....
Yardley	3	8	2.7
Bull Island	26	74	2.9
Easton	54	159	3.0
Belvidere	68	235	5.4
Delaware Water Gap	81	301	5.1
Four miles above Port Jervis	127	450	3.2
Lackawaxen	146	600	7.9
Deposit	212	984	5.8
Head	280	1,886	13.3

LEHIGH RIVER.

[From the census report on Water Power, 1880.]

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	159	.....
Bethlehem	12	205	3.8
Slatington	33	350	6.9
Lehighton	42	450	11.1
Mauch Chunk, below dam	46	504	13.5
	54	690	23.2
Near White Haven	70	1,105	25.9
Stoddartsville	83	1,457	27.1

SCHUYLKILL RIVER.

This river heads in Schuylkill County, in eastern Pennsylvania, and flows southeast to its junction with the Delaware at Philadelphia. In its course through Schuylkill and Berks counties it traverses numerous water gaps through the ridges of the Appalachian Valley. Its length is 112 miles, and its drainage basin about 1,800 square miles. The average fall of the river is about 5 feet per mile.

The profile is from the census report on Water Power, 1880.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	0	.....
Fairmount	8	0	.....
Pawlings dam	30	67	3.0
Reading	70	205	3.45
Port Clinton	92	390	8.4
Landingville	100	472	10.25
Schuylkill Haven	105	509	7.4
Port Carbon	112	619	15.7

SUSQUEHANNA RIVER.

Susquehanna River rises in two large branches known as the North and West branches, the former rising in southern New York and the latter in central Pennsylvania, both branches heading in the Allegheny plateau. The drainage basin of the entire river is 27,655 square miles, three-fourths of it being within the State of Pennsylvania. The length of the river, following up the North Branch, is 422 miles. The North Branch pursues an extremely crooked course, flowing in a generally southwesterly course in the State of New York, changing in northeastern Pennsylvania to a southeast course, until it flows out of the plateau and into the great Appalachian Valley. Then it flows southwestward to its junction with the West Branch at Sunbury. Most of this part of its course lies in limestone valleys, but in several cases it breaks through ridges from valley to valley, forming water gaps.

The West Branch heads in Cambria County and flows, at first, northward and northeastward and then southeastward until it leaves the plateau and enters the valley. Then it flows eastward in a limestone valley, at the south foot of the Allegheny Front, until it reaches a point north of Sunbury, whence it flows southward to its junction at that place with the North Branch, cutting several water gaps through opposing ridges on its way.

From Sunbury to Harrisburg the main Susquehanna flows nearly south across the trend of the ridges and valleys. In this part of its course the river cuts five fine water gaps. Through each of these the stream runs with accelerated velocity, while in the intervening limestone valleys it has a leisurely flow. From Harrisburg the river flows southeast to the head of Chesapeake Bay, most of the way through a rolling, highly cultivated country.

Between Sunbury and Harrisburg the profile is slightly convex, showing the effect in retarding the progress of the river's erosion of the hard ridges which it crosses.

The profile is from the census report on Water Power, 1880.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	0	.....
State line	12	69	5.8
Mouth of Fishing Creek	20	100	3.9
Foot of Columbia dam	43	224	5.4
Crest of Columbia dam	43	231	.....
Foot Conewago falls	57	254	1.6
Above Conewago falls	59	273	9.5
Harrisburg	69	298	2.5
Rockville	75	305	1.16
Clark ferry dam, foot	84	336	3.4
Clark ferry dam, crest	84	343	.....
Liverpool	99	378	2.3
Selinsgrove	116	421	2.5
Sunbury dam, foot	122	422	0.2
Sunbury dam, crest	122	429	.....
Nanticoke dam, foot	174	509	1.5
Nanticoke dam, crest	174	515	.....
Wilkesbarre	183	521	0.66
Mouth of Lackawanna River	190	536	2.14
Mouth of Tunkhannock Creek	211	581	2.14
Mouth of Meehoopany Creek	223	604	1.9
Mouth of Wyalusing Creek	244	646	2.0
Mouth of Wysox Creek	258	687	2.9
Towanda	262	700	3.2
Athens	278	744	2.7
Otsego Lake	422	1,193	3.1

WEST BRANCH OF SUSQUEHANNA RIVER.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	429	.....
Lewisburg dam, foot	7	431	.....
Lewisburg dam, crest	7	434	.....
Muncy dam, foot	23	462	1.75
Muncy dam, crest	23	469	.....
Williamsport dam, foot	39	498	1.8
Williamsport dam, crest	39	508	.....
Lockhaven dam, foot	65	539	1.2
Lockhaven dam, crest	65	550	.....
Queens Run dam, foot	69	551	0.25
Queens Run dam, crest	69	557	.....
Keating	105	695	3.8
Curwinsville	160	1,117	7.7

JUNIATA RIVER.

This is a large western branch of Susquehanna River, which heads in the Appalachian Valley, under the Allegheny Front, and flows with a generally eastward, but an extremely crooked, course to its mouth, a few miles above Harrisburg. Its course is throughout an alternation of gentle stretches in limestone valleys and of rapid courses through water gaps. Its drainage basin comprises 3,223 square miles.

The profile is from the census report on Water Power.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	336	.....
Millerstown dam, foot	16	380	2.7
Millerstown dam, crest	16	388	.....
Lewistown dam, foot	44	442	1.9
Lewistown dam, crest	44	450	.....
Newton Hamilton dam, foot	68	512	2.6
Newton Hamilton dam, crest	68	520	.....
Huntington dam, foot	90	610	4.1
Huntington dam, crest	90	622	.....

POTOMAC RIVER.

This river heads in several long branches in the Appalachian Valley, in Virginia, and in the edge of the Allegheny Plateau. The North and South branches of the Potomac and the Shenandoah River flow in lines that are parallel to one another, with a course somewhat east of north, for long stretches through limestone valleys. The North Branch, which is the uppermost of the head streams, turns eastward at Cumberland and, thence known as the Potomac, flows in a generally southeasterly course as far as Washington, where it reaches tide level, being joined on the way by the South Branch of the Potomac at Piedmont and by the Shenandoah River at Harpers Ferry. In the upper part of its course it crosses a number of ridges in water gaps, and at Harpers Ferry crosses the Blue Ridge, cutting a gap nearly 1,000 feet in depth.

The obstruction produced by the rocks of the Blue Ridge at Harpers Ferry has retarded the stream to such an extent that above this point both the main river and its branch, the Shenandoah, have been locally graded for a long distance upstream, giving them gentle and very crooked courses. The drainage basin of the Potomac River is 14,479 square miles, including the Shenandoah, which has an area of 2,850 square miles.

The following profiles of the Potomac, its South Branch, and the Shenandoah with its North Fork, are from the atlas sheets of the United States Geological Survey:

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth, Point Lookout	0	0	.....
Chain bridge, above Georgetown, D. C	115	20	0.2
Head of Little falls	116	40	20.0
Foot of Great falls	121	60	4.0
Head of Great falls	125	150	22.5
	141	200	3.1
Harpers Ferry	172	250	1.6
	179	300	7.1
Hancock	220	400	2.4
	244	500	4.2
Cumberland	290	600	2.2
	310	700	5.0
	316	800	16.7
	320	900	25.0
	324	1,000	25.0
	330	1,200	33.3

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Cumberland	334	1,400	50.0
	338	1,600	50.0
	342	1,800	50.0
	346	2,000	50.0
Wilson	358	2,500	41.7

SOUTH BRANCH OF POTOMAC.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth, near Green Spring	0	545	.....
	15	600	3.7
Romney	27	700	8.3
Moorefield	52	800	4.0
South Fork of South Branch	65	1,000	15.4
	76	1,200	18.2
	87	1,500	27.3
	103	2,000	31.3

SHENANDOAH RIVER.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth, Harpers Ferry	0	250	.....
	10	300	5.0
	39	400	3.4
Riverton	58	465	3.4
	68	500	3.5
Overall	82	600	7.1
	94	700	8.3
	115	800	4.8
	133	900	5.5
	147	1,000	7.1
	170	1,200	8.7
	200	1,500	10.0

NORTH FORK OF SHENANDOAH RIVER.

Locality.	Distance from mouth of Shenandoah River.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Riverton	58	465	.....
	63	500	7.0
	73	600	10.0
	83	700	10.0
	98	800	6.7
	124	900	3.8
	135	1,000	9.1
	147	1,200	16.7

JAMES RIVER.

This tributary to Chesapeake Bay heads in the Allegheny Front, in West Virginia, and flows in a generally eastward course across the Appalachian Valley, cutting across its ridges and finally the Blue Ridge. In its course across the valley and through the Piedmont region it has many rapids. At Richmond it crosses the Fall Line with an abrupt descent of 84 feet and just below that city reaches tide and the head of navigation. From this point to its mouth, 111 miles, it is a tidal estuary. Its drainage basin, including that of its main branch, Appomattox River, comprises 9,684 square miles.

Appomattox River heads in the Piedmont region, flows eastward and joins James River just below Petersburg, at which point it crosses the fall line.

The profile of James River is from the levels of the Engineer Corps, U. S. A., and that of the Appomattox from the atlas sheets of the United States Geological Survey.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	0	.....
Richmond	111	0	.....
Grant dam	114	84	28.0
Bosher dam	122	124	5.0
Maiden's Adventure dam	140	143	1.1
Tye River dam	220	375	2.9
Joshua Falls dam	246	463	3.4

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Lynchburg waterworks dam	260	513	3.6
Judith dam	263	540	9.0
Bald Eagle dam	268	558	3.6
Pedlar dam	271	572	4.7
Coleman Falls dam	274	588	5.3
Big Island dam	278	606	4.5
Cushaw dam	282	649	10.8
Blue Ridge dam	287	706	11.4
Quarry Falls dam	290	720	4.7
Varney Falls dam	297	759	5.6
Indian Rock dam	301	786	6.8
Wasp Rock dam	305	812	6.5
Junction Jackson and Cowpasture rivers	325	1,014	10.1

APPOMATTOX RIVER.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	0	.....
	17	100	5.8
Giles mill	59	200	2.4
Near Farmville	98	300	2.6
	110	400	8.3
Head near Appomattox	130	800	20.0

ROANOKE RIVER.

The source of this river is in the Appalachian Valley, in southwest Virginia; thence it flows southeastward to the head of Albemarle Sound. It is navigable to Weldon, above which place it crosses the Fall Line. Its principal branch, Dan River, which heads in the mountains of western North Carolina, exhibits similar peculiarities.

The drainage basin of Roanoke River, including that of the Dan, is 9,237 square miles.

The profile of the Roanoke as far up as Brookneal and that of the Dan are from levels by the Engineer Corps, U. S. A. The upper portion of the Roanoke is from the atlas sheets of the United States Geological Survey.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	0	.....
Weldon	129	39	0.3
Gaston	132	143	34.6
Stone House Creek	148	181	2.4
Great Creek	160	194	1.1
Allen Creek	172	212	1.5
East Lynne Creek	182	233	2.1
Clarksville, Va	196	289	4.0
Roanoke landing	217	319	1.4
Wallace Creek	223	328	1.5
Old landing	229	337	1.5
Edmunds landing	233	341	1.0
Coles ferry landing	239	350	1.5
Brookneal	249	368	1.8
Head Long Island	261	400	2.7
Wards road ferry	276	500	6.7
Folers ferry, mouth of Pig River	302	600	3.8
Radford ford	317	700	6.7
Lynville ford	330	800	7.7
	341	900	9.1
Salem	360	1,000	5.3
	365	1,100	20.0
	370	1,200	20.0
	379	1,300	11.1
	386	1,400	14.3
	388	1,500	50.0

DAN RIVER.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	289	.....
Danville canal	60	397	1.8
Wilson upper ferry	67	431	4.9
Daniels ferry	78	462	2.8
Leaksville	91	492	2.3
Eagle falls	99	510	2.3
Madison bridge	110	538	2.5
Ladds ford	119	568	3.3
	128	618	5.6
Danbury	138	688	7.0

CAPE FEAR RIVER.

This river heads in the upper part of the Piedmont region and flows southeast to its mouth at Wilmington. It crosses the Fall Line above Fayetteville.

Its drainage basin comprises 8,310 square miles.

The profile is from the report on Water Power of the Tenth Census.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Wilmington	0	0	.....
Fayetteville	112	7	.06
Smiley falls, foot	139	42	1.3
Smiley falls, head	143	69	6.75
Junction Haw and Deep rivers	172	130	2.1

GREAT PEDEE RIVER.

This river, following up the Yadkin, heads in the Blue Ridge, in the northern part of North Carolina, and flows southeastwardly to its mouth, near Georgetown, South Carolina, its course being throughout in the Piedmont region and the Atlantic Coastal Plain. It exhibits many irregularities, due to recent tilting of the region through which it flows. It crosses the Fall Line above Cheraw.

Its drainage basin is 17,098 square miles.

The profile is from the levels of the Engineer Corps, U. S. A., as far as Wilkesboro; above that point it has been derived from the atlas sheets of the United States Geological Survey.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	0	.....
Cheraw	149	65	0.4
Foot of Bluitt falls	170	145	3.8
Allentown ferry	199	201	1.9
Christian ferry	208	209	0.9
Mouth Uwharrie River	212	239	7.5
Head of Narrows	214	259	10.0
Stokes ferry	227	505	18.9
Bringle ferry	237	557	5.2
Crossing Southern R. R	259	661	4.7
Mouth of South Yadkin River	263	667	1.5
Dutchman Island	272	680	1.4
Oakes ferry	281	694	1.6

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Bailey ferry	291	711	1.7
Hall ferry	297	726	2.5
Jones mill	309	752	2.2
Sycamore ford	325	790	2.4
Rockford	341	866	4.8
Hurt ledge	351	908	4.2
Sayles ford	362	931	2.1
Mouth of Roaring River	371	958	3.0
Wilkesboro	382	992	3.1
	385	1,000	2.7
	391	1,100	16.7
	408	1,200	8.3
Patterson	410	1,250	7.1

SANTEE RIVER.

This river heads in the eastern slope of the Blue Ridge in several branches, converging in the Catawba, Saluda, and Broad rivers, which flow southwestward across the Piedmont region and the Atlantic Coastal Plain to its mouth, near Cape Romain. Its drainage basin is 14,696 square miles.

The profiles of this river and its branches are from the levels of the Engineer Corps, U. S. A.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	0	------
Junction of Wateree and Congaree rivers	208	80	0.4
Head of Wateree River	268	160	1.3
Foot of Gaydon falls	275	226	9.4
Crossing C. C. and A. R. R.	308	461	7.1
State Line	323	485	1.6
Crossing A. and C. R. R.	328	508	4.6
Crossing C. C. R. R	333	522	2.8
Crossing W. N. C. R. R.	370	730	5.6
Crossing Chester and Lenoir R. R	398	884	5.5
Moore shoal	403	908	4.8
Morganton	413	1,010	10.2
Old Fort	463	1,274	5.3
Crossing of Southern R. R.	473	2,050	77.6
Head, in Swannanoa gap	479	2,658	101.3

CONGAREE RIVER.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Junction with Wateree River	0	80	.....
Head, Columbia	60	136	1.0

SALUDA RIVER.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	136	.....
Crossing of G. & C. R. R	60	383	4.1
	125	749	5.6
Crossing of Southern R. R	135	809	6.0

BROAD RIVER.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	136	.....
Ninety-nine Islands	12	176	3.3
Summers shoal, foot	26	229	3.7
Lyle shoal, foot	41	270	2.7
Neal shoal, foot	59	322	2.9
Foot of the gravel	68	340	2.0
Foot of Ninety-nine Islands	94	426	3.3
Head of Cherokee shoal	101	530	14.8
Crossing Southern R. R	105	542	3.0
Green River	141	759	6.0

SAVANNAH RIVER.

This river heads in the eastern slope of the Blue Ridge and flows southeast to its mouth at Savannah. It crosses the Fall Line just above Augusta, where is afforded a fine water power, which is utilized to a large extent, making Augusta the chief cotton-manufacturing point in the South.

The drainage basin of the river is 11,402 square miles.

The profile is from levels by the Engineer Corps, U. S. A.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Savannah	0	0	.....
Crossing of C. & S. R. R.	14	4	0.3
Purysburg	23	6	0.25
Ebenezer landing	34	14	0.7
Sisters ferry	47	20	0.5
Parachuchla	55	24	0.5
Monkey point	70	31	0.5
Haga Slaga point	91	44	0.6
Cohens bluff	101	51	0.7
Burton ferry landing	116	60	0.6
Brown landing	130	69	0.6
Rattlesnake camp	141	77	0.7
Blue bluff	150	80	0.3
Demerief ferry	161	87	0.6
Eagle point	170	92	0.6
Hill landing	180	98	0.6
Mason landing	191	103	0.5
Augusta	203	110	0.6
Little River shoals	225	203	4.2
Point Lookout shoals	235	256	5.3
Petersburg, Ga	259	289	1.4
	265	306	2.8
Head Hell sluices	272	380	10.6
Moseley ferry	278	405	4.2
Harper ferry	284	422	2.8
Dooley ferry	306	541	5.4
Andersonville	314	577	4.5

OCONEE RIVER.

This is one of the main branches of the Altamaha River in Georgia. It heads on the eastern slope of the Blue Ridge and flows nearly southeast to its junction with the Altamaha. Its profile is remarkably smooth, with little variation from a normal one. It is from levels by the Corps of Engineers, U. S. A.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	83	.....
Chaney ferry	6	88	0.8
McArthur landing	15	99	1.2
Adams landing	25	109	1.0
Savannah, Americus and Montgomery R. R. crossing	30	114	1.0
Adams landing	37	121	1.0
Odam landing	39	123	1.0
Dixon landing	45	129	1.0
Walton landing	50	134	1.0
Davis landing	55	138	0.8
Branch landing	58	141	1.0
Cooper landing	62	145	1.0
Walton landing	67	151	1.2
Pritchett landing	70	154	1.0
Clark landing	72	156	1.0
Dublin	79	161	0.7
Blackshears ferry	85	167	1.0
Kittrell landing	93	175	1.0
Thompson ferry	98	179	0.8
Ball ferry	102	185	1.5
Central R. R. bridge	108	193	1.3
Spring Lake	114	202	1.5
Whitaker Island	127	221	1.5
Tucker ferry	137	231	1.0
Milledgeville	147	242	1.1
Georgia R. R. bridge	149	248	3.0
Crossing Georgia R. R	209	335	1.5
Crossing N. E. R. R. near Athens	249	604	6.7
Crossing N. E. R. R. near Lula	294	1,232	13.9

APALACHICOLA AND CHATTAHOOCHEE RIVERS.

The Chattahoochee, the main branch of the Apalachicola River, heads in the Blue Ridge, in northeast Georgia, flows southwest to the boundary between this State and Alabama, following that boundary to the south line of the State, near which it joins Flint River to form the Apalachicola. The river leaves the metamorphic region and crosses the Fall Line at Columbus, the head of navigation. Just above this place is a succession of rapids and falls, giving the river a descent of 120 feet within 5 miles of Columbus.

The profile is taken mainly from the atlas sheets of the United States Geological Survey.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth of Apalachicola River	0	0	.....
Columbus	400	238	0.6
	405	358	24.0
West Point	434	600	8.3
Morns ferry	512	700	1.3
Lizzie	537	750	2.0
	554	800	2.9
Terry ferry	586	900	3.1
Stringer ford	615	989	3.1
Crowder ford	627	1,100	9.3
	643	1,200	6.25
Mouth of Spoiled Cane Creek	653	1,500	30.0
Head	661	3,500	250.0

COOSA AND TALLAPOOSA RIVERS.

Alabama River, one of the two main branches of Mobile River, divides just below Montgomery, Alabama, into two large branches, Coosa and Tallapoosa rivers. The former heads in northern Georgia, in the broken hills which form the southern end of the Appalachian Mountains in two branches, the Etowah and the Oostanaula rivers. The Tallapoosa River heads in northwest Georgia.

The entire drainage basin of the Alabama is 23,820 square miles.

The profile is from levels of the Engineer Corps, U. S. A.

COOSA RIVER.

Locality.	Distance from Wetumka bridge.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Wetumka bridge	0	148	.....
Thompson ferry	19	244	5.1
Zimmerman ferry	31	276	2.7
Narrows	46	383	7.1
Foot of Steamboat Island	58	406	1.9
Bullock Island	64	413	1.2
Tallasseehatchie Creek	76	415	0.2
Glover ferry	85	423	0.9
Griffith ferry	96	453	2.7
Truss ferry	108	469	1.3
Embry ferry	115	471	0.3
Greensport	141	500	1.1
	160	521	1.1
Garrett ferry	207	571	1.1
Cedar bluff	223	623	3.3
Rome	271	652	0.6

ETOWAH RIVER.

[This profile is made up mainly from the atlas sheets of the United States Geological Survey.]

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth, Rome	0	652	.....
Cartersville	43	696	1.0
Mouth of Little River	63	798	5.1
Chamlee ferry	83	900	5.1
Ledbetter bridge	104	1,000	4.8
Dougherty	119	1,100	6.7
	122	1,200	33.3
	131	1,300	11.1
Head	144	3,500	169.2

OOSTANAULA RIVER AND COOSAWATTEE RIVER.

[This profile is made up from atlas sheets of the United States Geological Survey.]

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth, Rome	0	652	.....
Foot of mountains	67	700	0.7
	73	1,000	50.0
	83	1,200	20.0

BLACK WARRIOR RIVER.

This is a large eastern branch of Tombigbee River, one of the two forks of Mobile River. It heads in northern Alabama and flows southwest to its junction with the Tombigbee.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Tuscaloosa	0	0	.....
Langston	13	45	3.5
Patton ferry	32	107	3.3
Mouth of Locust Fork	47	120	0.9
Tuggle landing	70	144	1.0
Junction Sipsey and Mulberry forks	90	163	0.95

TALLAPOOSA RIVER.

[This profile is made from atlas sheets of the United States Geological Survey.]

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	160	.....
Bosworth	90	500	3.8
Malone ferry	106	582	5.1
Mouth of Little Tallapoosa River	122	660	4.9
	148	800	5.4
Muscadine	181	900	3.0
	200	1,000	5.3
Allgood mill	215	1,154	10.3

TEXAS RIVERS.

The rivers of Texas, the Trinity, Brazos, Colorado, and Nueces, and their branches, drain a territory whose physiographic features affect all its streams in a similar manner. The region slopes with much uniformity toward the southeast and the streams flow down this slope. It is formed of a succession of rock beds, dipping gently southeast, but more steeply than the slope of the land, so that on following down one of these streams we cross successively more recent strata. These beds are not of uniform hardness, and the streams, cutting more rapidly in the softer beds and being retarded by the harder ones, exhibit a succession of gentle and rapid courses. This is excellently shown in the Brazos and Colorado rivers.

These profiles were made up from atlas sheets of the United States Geological Survey.

TRINITY RIVER.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	0	.....
Crossing of H. E. & W. T. R. R	72	50	0.7
Sixteen miles below Dallas	314	350	1.2
Twelve miles above Dallas	342	400	1.8
Six miles below Fort Worth	361	500	5.3
Weatherford	404	1,000	11.6

BRAZOS RIVER AND CLEAR FORK.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	0	.....
Bolivar landing	51	2	.....
Mouth of Cow Creek	60	9	0.8
Crossing G. C. & S. F. R. R	66	22	2.2
Richmond	89	47	1.1
Eighteen miles above Waco	291	400	1.7
	318	450	1.85
	341	500	2.2
	374	600	3.0
	407	700	3.0

BRAZOS RIVER AND CLEAR FORK—continued.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Near Palo Pinto	462	800	1.8
	521	900	1.7
	545	1,000	4.2
	572	1,100	3.7
Near Fort Griffin	592	1,200	5.0
	621	1,400	6.2
	642	1,500	4.8
Newsom	675	1,650	4.5
Roby	710	1,900	7.1

COLORADO RIVER.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	0	--------
	145	300	2.1
Above Bastrop	175	350	1.7
Below Austin	205	400	1.7
Dam above Austin	217	450	4.2
	229	500	4.2
	261	600	3.1
Marble Falls	297	700	2.8
Mouth of Llano River	315	800	5.6
	329	900	7.1
	341	1,000	8.3
	371	1,100	3.3
	424	1,200	1.9
	468	1,300	2.3
	498	1,400	3.3
	524	1,500	3.8
	548	1,600	4.2
	569	1,700	4.7
	591	1,800	4.5

LLANO RIVER.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	800	.....
Llano	27	1,000	7.4
	48	1,200	9.5
	84	1,500	8.3

SAN SABA RIVER.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	1,120	.....
Above San Saba	14	1,200	5.7
	37	1,400	8.7
	61	1,600	8.3
Fort McKavett	111	2,100	10.0

CONCHO RIVER.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	1,500	.....
Above Paint Rock	22	1,600	4.5
	30	1,700	12.5
San Angelo	43	1,800	7.7
	55	1,900	8.3
	66	2,000	9.1
	83	2,200	11.7

NUECES RIVER.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	0	.....
Northern part of Zavalla County	230	700	3.1
	256	1,000	11.5
Northwestern part of Uvalde County	275	1,250	13.2
Barksdale	292	1,500	14.7
	310	1,750	13.8

RIO GRANDE.

The Rio Grande rises in the San Juan Mountains of Colorado and has a steep descent in a mountain canyon to San Luis Valley. In this broad valley its slope is greatly reduced, but increases again in the canyon in southern Colorado and northern New Mexico, which it has cut in a volcanic mesa. Flowing out of this it runs southward for several hundred miles in a valley, in which its slope is fairly uniform. It increases again in the canyon above the mouth of the Pecos, diminishing again below that point toward its mouth, where it becomes very slight.

The river is subject to wide variations in volume at different times of the year. From southern New Mexico to the mouth of the Pecos it is often dry in late summer, while in the springtime it is a powerful torrent.

The Pecos, its main branch, heads in the mountains above Las Vegas and flows southeastward over the plains and through a broad valley to its mouth. Its profile, although evidently controlled largely by the slopes of the surface, is nearly normal, but shows a decided irregularity in the increased slope near its mouth.

The profile of the Rio Grande has been made up mainly from the atlas sheets of the United States Geological Survey, that of the Pecos in part from railroad levels and in part from these atlas sheets.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	0	--------
	250	440	1.8
	420	960	3.1
	722	2,500	5.1
	765	2,600	2.3
	783	2,700	5.6
	798	2,800	6.7
	816	2,900	5.6
	829	3,000	7.7
	843	3,100	7.1
	878	3,200	2.8
	902	3,300	4.1
	916	3,400	7.1
	956	3,500	2.5
	992	3,600	2.7
El Paso	1,030	3,700	2.6

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Earlham	1,069	3,800	2.6
	1,099	3,900	3.3
	1,153	4,120	4.1
Alamocita	1,197	4,250	2.9
San Marcial	1,233	4,400	4.2
Albuquerque	1,374	4,900	3.5
Bernalillo	1,396	5,000	4.5
Cochiti	1,420	5,200	8.3
San Ildefonso	1,450	5,500	10.0
Rinconada	1,480	5,829	10.9
	1,508	6,357	18.9
Colonas ferry	1,552	7,443	24.7
Del Norte	1,626	7,742	4.0
Wagon Wheel gap	1,656	8,450	23.6
San Juan	1,681	9,000	22.0
Mouth of Lost Trail Creek	1,756	9,500	6.7
Mouth of Pole Creek	1,765	10,790	143.3
Head	1,770	12,000	242.0

PECOS RIVER.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	1,000	.....
Pecos	215	2,550	7.2
Eddy	300	3,107	6.6
Hagerman	375	3,400	3.9
Crossing P. V. R. R.	405	3,500	3.3
Latitude 35°	530	4,600	8.8
Las Colonias	548	4,850	13.9
La Junta	556	4,950	12.5
	567	5,100	13.6
Anton Chico	577	5,250	15.0
La Cuesta	597	5,800	27.5
San Miguel	607	6,000	20.0
	622	6,500	33.3
	637	7,000	33.3
	657	8,000	50.0
Head	663	12,000	666.7

MISSISSIPPI RIVER.

The Mississippi River heads in Lake Itasca, or rather in the indefinite divide a few miles farther south. For several hundred miles its course is through a region over which the great northern glacier scattered its deposits, producing great irregularity in the disposition of the drainage. In this part of its course the river is very irregular, both in direction and in slope, flowing through numerous lakes of considerable size, and between them having rapids and falls. The most notable of the falls in the upper part of its course is that of St. Anthony, at Minneapolis, where the river flows off a bed of St. Peter sandstone. Below this fall its course becomes much more regular. It flows through Lake Pepin, where it has no appreciable descent, and there are no rapids of importance until those of Rock Island, Illinois, are reached, and these, with the rapids at Keokuk, are the only serious obstructions to navigation below Minneapolis. In each of these cases the rapids were produced by a recent shift in the river channel, caused by the Laurentian glacier.

The slope of the river as far as Cairo does not vary greatly, ranging from two feet to eight-tenths of a foot per mile. Most of the way the river flows in a bottom land of considerable breadth, generally widening downward, and the river becomes more and more crooked and winding.

Below Cairo, where it is joined by the Ohio, its character is that of a graded stream. Although its slope is not materially diminished, its bottom lands become broader, and within them it meanders widely from bluff to bluff. In southern Louisiana it changes again; its course becomes straighter, it deposits more detritus than it receives, and in consequence it has built up its bank by overflow, forming a broad, gently sloping ridge of dry land which accompanies it through the swamp in which its course lies. Below the mouth of Red River it forms distributaries which aid in carrying off its surplus water in times of flood. In this part of its course its slope is scarcely appreciable, and yet it flows with a strong current.

It discharges into the gulf through a delta composed of several arms or passes—South Pass, Southwest Pass, Pass a l'Outre, etc. Its entire drainage basin comprises 1,240,039 square miles.

The profile is from the levels of the Mississippi River Commission; the heights are those of ordinary low water.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	0	.....
Carrollton, La	140	1	.....
College Point, La	194	2	.....
Plaquemine, La	222	2	.....
Baton Rouge, La	245	3	.....
Port Hickory, La	287	3	.....
Mouth of Red River, Ark	316	7	0.0
Natchez, Miss	378	21	0.2
St. Joseph, La	449	36	0.2
Vicksburg, Miss	487	48	0.3
Lake Providence, La	555	73	0.4
Greenville, Miss	618	93	0.3
Arkansas City, Ark	658	100	0.2
Mouth of White River, Ark	700	114	0.3
Helena, Ark	791	147	0.4
Mahoons landing, Miss	821	163	0.5
Memphis, Tenn	862	185	0.5
Fulton, Tenn	922	212	0.5
Cottonwood Point, Mo	974	230	0.3
New Madrid, Mo	1,028	256	0.5
Columbus, Ky	1,076	271	0.3
Cairo, Ill	1,097	274	0.1
Thebes, Ill	1,137	291	0.4
Fountain Bluff, Ill	1,177	313	0.5
St. Louis, Mo	1,270	380	0.7
Mouth of Missouri River, Mo	1,288	395	0.8
Grafton, Ill	1,308	405	0.5
Clarksville, Mo	1,361	433	0.5
Louisiana, Mo	1,371	437	0.4
Hannibal, Mo	1,397	450	0.5
Quincy, Ill	1,413	458	0.5
Canton, Mo	1,428	466	0.5
Gregory landing, Mo	1,439	472	0.5
Alexandria, Mo	1,445	475	0.5
Keokuk, Iowa	1,450	477	0.4
Montrose, Iowa (rapids)	1,462	500	1.9
Fort Madison, Iowa	1,470	502	0.3
Burlington, Iowa	1,492	511	0.4
Oquawka, Ill	1,505	516	0.4
Keithsburg, Ill	1,516	523	0.6
New Boston, Ill	1,522	524	0.2

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Port Louisa, Iowa	1,530	526	0.3
Muscatine, Iowa	1,544	530	0.3
Rock Island, Ill	1,571	542	0.4
Le Claire, Iowa (rapids)	1,587	562	1.2
Clinton, Iowa	1,608	566	0.2
Savanna, Ill	1,629	572	0.3
Bellevue, Iowa	1,649	578	0.3
Dubuque, Iowa	1,672	585	0.3
Cassville, Wis	1,699	599	0.5
Prairie du Chien, Wis	1,727	604	0.2
Lansing, Iowa	1,756	612	0.3
Brownsville, Minn	1,782	622	0.4
La Crosse, Wis	1,791	628	0.7
Winona, Minn	1,821	637	0.3
Fountain, Wis	1,829	644	0.9
Minneiska, Minn	1,839	650	0.6
Alma, Wis	1,849	656	0.6
Wabasha, Minn	1,857	662	0.8
Red Wing, Minn., Lake Pepin	1,888	665	0.1
Hastings, Minn	1,911	671	0.3
St. Paul, Minn	1,937	683	0.5
Mouth of Minnesota River, Minn	1,943	688	0.8
Minneapolis, Minn. (St. Anthony falls)	1,952	794	11.8
Anoka, Minn	1,970	825	1.7
Mouth of Elk River, Minn	1,985	851	1.7
Monticello, Minn	1,997	891	3.3
Clearwater, Minn	2,015	936	2.5
St. Cloud, Minn	2,025	965	2.9
Watab, Minn	2,033	1,001	4.5
Mouth of Platte River, Minn	2,045	1,026	2.1
Little Falls, Minn	2,060	1,090	4.3
Mouth of Crow Wing River, Minn	2,085	1,145	2.2
Brainerd, Minn	2,095	1,150	0.5
Aitkin	2,128	1,190	1.2
Foot Grand Rapids	2,182	1,248	1.1
Mouth of Leech Lake River	2,210	1,281	1.2
Lake Winnebagoshish	2,222	1,290	0.8
Lake Pemidgi	2,263	1,355	1.6
Lake Itasca	2,296	1,462	3.2

OHIO RIVER.

Ohio River drains nearly all of the Allegheny Plateau from New York to Alabama. Its two head branches, the Allegheny and Monongahela, head respectively in southern New York and in West Virginia, and both these streams have a rapid descent to their point of junction at Pittsburg. From that point to Cairo, the mouth of the Ohio, the slope of the river is gentle, being, on an average, much less than that of the Mississippi between Minneapolis and Cairo. Its long course of over 963 miles is broken by only one fall of importance, that at Louisville. Its drainage basin, including all its branches, is 201,720 square miles.

Its branches upon the left come out of the Allegheny Plateau. Of these streams one, however, Kanawha River, heads far to the southeast of the plateau among the high mountains of western North Carolina, and after a tortuous and irregular course in these mountains and among the valley ridges under the name of New River, it crosses the Allegheny Front and cuts a tremendous gorge through the plateau. Just below its junction with the Gauley, its principal tributary, it flows over Kanawha Falls, caused by the presence of a bed of hard sandstone, lying nearly horizontal. From the foot of these falls its slope is greatly reduced, so that it is navigable to this point. The Greenbrier and Gauley, which are its main tributaries, have steep and irregular profiles. The Guyandot, Big Sandy, and Little Kanawha rivers, all of which are tributaries of the Ohio on the left, head in the Allegheny Plateau and flow down canyons which they have cut in it.

The profiles of the Ohio, Allegheny, and Clarion are from levels by the Corps of Engineers, U. S. A. That of the Ohio refers to ordinary low water.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Cairo, Ill	0	277	.....
Hillerman, Ill	27	281	0.1
Paducah, Ky	47	284	0.2
Smithland, Ky	60	286	.....
Bay City, Ill	70	287	0.1
Golconda, Ill	80	290	.....
Rosiclare, Ill	91	290	0.1
Weston, Ky	107	295	.....
Caseyville, Ky	111	297	.....
Shawneetown, Ill	123	301	0.3
Raleigh, Ind	130	302	.....
Uniontown, Ky	140	306	.....

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mount Vernon, Ind	153	308	0.2
West Franklin, Ind	165	312	.....
Henderson, Ky	178	316	0.3
Evansville, Ind	190	318	.....
Newburg, Ind	204	320	0.2
Enterprise, Ind	219	326	.....
Owensboro, Ky	226	328	0.4
Rockport, Ind	235	330	.....
Grandview, Ind	240	331	.....
Lewisport, Ky	244	333	.....
Troy, Ind	252	335	0.3
Tell City, Ind	356	337	.....
Hawsville, Ky	260	348	.....
Cloversport, Ky	272	340	0.3
Stevensport, Ind	283	340	.....
Derby, Ind	292	343	.....
Concordia, Ky	297	346	0.2
Reno, Ind	301	346	.....
Alton, Ind	305	347	.....
Leavenworth, Ky	319	349	.....
New Amsterdam, Ind	327	353	0.2
Mauckport, Ind	336	354	.....
Brandenburg, Ky	338	356	.....
New Albany, Ind	374	367	0.3
Louisville, Ky	378	394	6.7
Utica, Ind	386	395	.....
Herculaneum	396	399	.....
Bethlehem, Ind	405	399	0.2
Madison, Ind	423	401	.....
Carrollton, Ky	435	404	0.2
Vevay, Ind	443	408	.....
Florence, Ind	452	411	.....
Warsaw, Ky	454	411	.....
Patriot, Ind	462	413	0.3
Rising Sun, Ind	474	420	.....
Aurora, Ind	482	425	.....
Lawrenceburg, Ind	486	425	0.5
Taylorsville, Ky	498	429	.....
Anderson Ferry, Ky	504	429	.....
Cincinnati, Ohio	511	431	0.2
New Palestine, Ohio	528	437	.....

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
New Richmond, Ohio	532	438	.....
Point Pleasant, Ohio	537	439	0.3
Moscow, Ohio	540	439	.....
Neville, Ohio	544	440	.....
Chilo, Ohio	549	441	.....
Utopia, Ohio	553	442	.....
Augusta, Ky	556	444	.....
Higginsport, Ohio	558	445	.....
Dover, Ohio	564	446	.....
Ripley, Ohio	566	447	.....
Maysville, Ky	576	448	0.2
Manchester, Ohio	587	451	.....
Wrightsville, Ohio	593	454	.....
Rome, Ohio	600	457	0.4
Buena Vista, Ohio	611	460	.....
Rockport, Ky	614	461	.....
Quincy, Ky	618	464	.....
Portsmouth, Ohio	630	468	0.4
Sciotoville, Ohio	638	472	.....
Greenup, Ky	651	478	.....
Ironton, Ohio	660	483	0.5
Ashland, Ky	665	486	.....
Ceredo, W. Va	672	490	.....
Haskelville, Ohio	690	501	0.6
Miller, Ohio	692	501	.....
Gallipolis, Ohio	717	511	0.4
Ravenswood, W. Va	744	544	.....
Portland, Ohio	749	546	1.1
Parkersburg, W. Va	780	564	0.6
Marietta, Ohio	792	570	0.5
Newport, Ohio	807	579	.....
New Matamoras, Ohio	822	588	.....
Sistersville, W. Va	826	590	0.6
Moundsville, W. Va	861	614	.....
Wheeling, W. Va	871	622	0.7
Burlington, Ohio	875	628	.....
Warrenton, Ohio	882	632	.....
Wellsburg, W. Va	889	635	.....
Steubenville, Ohio	896	641	0.8
Wellsville, Ohio	915	656	.....
East Liverpool, Ohio	920	657	0.7
Vanport, Pa	935	669	.....
Baden, Pa	942	673	0.7
Pittsburg, Pa	963	702	1.4

ALLEGHENY RIVER.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth, Pittsburg	0	696	.....
Sharpsburg	5	704	1.6
Pittsburg waterworks	7	710	3.0
Springdale	17	721	1.1
Tarentum	22	728	1.4
Freeport	30	744	2.0
Mouth of Mahoning River	55	797	2.1
Mouth of Red Bank River	64	821	2.7
Mouth of Clarion River	85	862	2.0
Mouth of East Sandy Creek	118	953	2.8
Mouth of French Creek	123	969	3.2

CLARION RIVER.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	862	.....
Big falls	3	882	6.7
Turnip Hole	12	932	5.6
Callensburg	18	992	10.0
Clarion	32	1,042	3.6
Mill Creek	38	1,062	3.3
State road	42	1,080	4.5
Hemlock	46	1,095	3.8
Cooksburg	48	1,114	9.5
Clarington	53	1,156	8.4
Millstone	58	1,181	5.0
Spring Creek	69	1,255	6.7
Peach Bottom	80	1,328	6.6
Portland shoals	86	1,378	8.3

ROUGH RIVER.

[From levels of the Corps of Engineers, U. S. A.]

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth, Livermore	0	0	.....
Hartford	28	17	0.6
Hedge River	45	31	0.8
Comb of Hines dam	55	44	1.3
Comb of Landrum dam	72	57	0.8
Comb of Greens dam	81	79	2.4
Frank mill	105	89	0.4
Lampton mill	114	101	1.3

BIG SANDY RIVER.

[From levels by the Corps of Engineers, U. S. A.]

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Catlettsburg	0	488	.....
White Creek landing	8	502	1.8
Turnam ferry	13	504	0.4
Louisa	26	515	0.9

LOUISA FORK.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Louisa	0	515	.....
Peach Orchard	17	539	1.4
Red House shoal	27	563	2.4
Paintville	36	576	1.4
Prestonburg	52	596	1.3
Lanesville	72	627	1.6
Piketon	87	649	1.5

TUG FORK.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Louisa	0	515	.....
Falls, head	11	538	2.1
Double shoal	20	551	1.4
Warfield	35	577	1.7

GUYANDOT RIVER.

[From levels by the Corps of Engineers, U. S. A.]

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	497	.....
Rogers mill	13	523	2.0
Salt Rock dam	25	539	1.3
Falls	31	549	1.7
Laurel shoals	42	562	1.2
Lambert mill	54	584	1.8
Big Creek shoal	64	599	1.5
Peck mill	75	627	2.5
Logan	82	645	2.6

KANAWHA AND NEW RIVERS.

[From levels by the Corps of Engineers, U. S. A.]

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth at Point Pleasant	0	510	.....
Eighteen-mile ripple	18	517	0.4
	25	524	1.0
	36	532	0.7
Near St. Albans	44	540	1.0
Tyler shoal	55	549	0.8
Charleston	58	555	2.0
Brownstown	68	557	0.2

KANAWHA AND NEW RIVERS—continued.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Cabin Creek	74	565	1.3
Paint Creek	80	572	1.2
Cannelton	85	585	2.6
Foot of Kanawha falls	95	617	3.2
Mouth of Gauley River	98	650	11.0
	117	1,000	18.4
	129	1,100	8.3
	143	1,200	7.1
	155	1,300	8.3
	172	1,400	5.9
Mouth of East River	193	1,500	4.8
Ripplemead	211	1,600	5.6
Churchwood	229	1,700	5.6
	253	1,800	4.2
Mouth of Reed Creek	280	1,900	3.7
	293	2,000	7.7
	299	2,100	16.6
	306	2,200	14.3
Daughten ford	330	2,300	4.2
Bridle Creek	348	2,400	5.6
Weaversford, forks	356	2,500	12.5
	376	2,600	5.0
	392	2,700	6.3

GAULEY RIVER.

[From the atlas sheets of the United States Geological Survey.]

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	650	.....
	10	700	5.0
	15	800	20.0
	19	900	25.0
	23	1,000	25.0
	26	1,100	33.3
	30	1,200	25.0
	33	1,300	33.3
	37	1,400	25.0
	40	1,500	33.3
	45	1,600	20.0
	49	1,700	25.0
	56	1,800	14.3
	60	1,900	25.0
	65	2,000	20.0
	74	2,100	11.1
	83	2,200	11.1
	93	2,300	10.0
	95	2,400	50.0
	99	2,500	25.0
Head	109	4,000	150

GREENBRIER RIVER.

[From the atlas sheets of the United States Geological Survey.]

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth, Hinton	0	1,350	.....
	5	1,400	10.0
	17	1,500	8.3
	31	1,600	7.1
Near Caldwell	48	1,700	5.9
	51	1,800	33.3
	69	1,900	5.6
	86	2,000	5.9
	97	2,100	9.1

LITTLE KANAWHA RIVER.

[From levels by the Corps of Engineers, U. S. A.]

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Parkersburg	0	564	.....
Leachtown	14	579	1.1
Elizabeth	26	601	1.8
Buffington shoals	44	613	0.7
Nailor bend	54	628	1.5
Mouth of Anna Maria Creek	64	640	1.2
Mouth of Pine Creek	76	655	1.3
Grantsville	78	661	3.0
Acre Island	86	672	1.4
Cedar Creek	96	688	1.6
Glenville	103	696	1.1
Stout mill	114	724	2.5
Lumberport	122	742	2.3
Bulltown	130	761	2.4

TENNESSEE RIVER.

Tennessee River heads in many branches, under a variety of topographic conditions, in the Appalachian Valley and in the mountains of western North Carolina. Certain of the heads of the Tennessee—the Hiwassee, the Little Tennessee with its branches, the Nantahala, Tuckaseegee, Big Pigeon, French Broad, and Nolichucky—rise in this region. Among the mountains their courses are steep, and they emerge into the Appalachian Valley through gorges cut in the Great Smoky range. Below these gorges their slopes are comparatively gentle.

Other of its head streams, including the Holston, with its branches, and the Clinch and Powell rivers, head in southwest Virginia in the Appalachian Valley, and their courses, for the most part, follow down secondary valleys on limestone formations, in which they have established gentle slopes. Here and there, however, these streams cut their way across valley ridges from one of these secondary valleys to its neighbor, and in these water gaps the uniformity of their slopes is interrupted and rapids and falls result.

Tennessee River is formed by the union of the Holston and French Broad rivers, a few miles above Knoxville. From the point of junction it flows southwest down the Appalachian Valley to Chattanooga. Here it turns abruptly westward, cutting through the Cumberland

Plateau. A few miles below, it receives the waters of Sequatchie River. This stream flows in a southwest course throughout, draining a portion of the plateau.

The Tennessee River below Knoxville has a gentle slope, averaging not more than a foot to a mile as far as Mussel shoals. Here the river flows over a succession of beds of hard limestone, which have retarded its work of erosion and given rise to rapids known as Big and Little Mussel and Colbert shoals. Below the latter the river has a very slight fall, averaging only about three-tenths of a foot to a mile.

The drainage basin of Tennessee River, including all its branches, is 43,897 square miles.

The profile of Tennessee River is from levels by the Corps of Engineers, U. S. A. Those of its branches, the Sequatchie, Hiwassee, Little Tennessee, Nantahala, Tuckaseegee, French Broad, Big Pigeon, Nolichucky, Holston, Powell, and Clinch rivers, have been made up from the atlas sheets of the United States Geological Survey.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Paducah	0	286	--------
Johnsonville	95	315	0.3
Riverton	225	358	0.3
Colbert shoals	233	384	3.3
Florence	255	396	0.6
Little Mussel shoals	259	418	5.5
Lock No. 1	275	503	5.3
Lock B	283	509	0.8
Milton bluff	285	523	7.0
Decatur	303	529	0.3
Guntersville	347	531	0.1
Bridgeport	402	593	1.1
Shellmound	412	598	0.5
The Skillet	436	606	0.3
The Suck	440	613	1.8
Chattanooga Creek	448	626	1.6
Chattanooga	452	631	1.3
Charleston	508	684	0.9
Rockwood	541	699	0.5
Kingston	556	712	0.9
Loudon	579	736	1.0
Knoxville	635	806	1.3
Head, junction Holston and French Broad rivers	639	810	1.0

SEQUATCHIE RIVER.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	595	.....
Near Dunlap, Tenn	32	700	3.3
	45	800	7.7
	61	900	6.25
	71	1,000	10.0

HIWASSEE RIVER.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	662	.....
Bean Mountain Canyon	36	700	0.9
Narrows	50	1,000	21.4
	54	1,200	50.0
Murphy	84	1,500	10.0
Hayesville	105	1,800	14.3
Above Hiwassee	116	2,000	18.2
Head	126	3,800	180.0

LITTLE TENNESSEE RIVER.

Locality.	Distance from mouth.	Height above. sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth at Lenoir, Tenn	0	750	.....
Morgantown	12	800	4.2
	35	900	4.3
	40	1,000	20.0
	43	1,100	33.3
	47	1,200	25.0
	55	1,300	12.5
	65	1,400	10.0
Mouth of Tuckaseegee River	71	1,500	16.6
Mouth of Nantahala River	77	1,550	8.3
	82	1,700	30.0
Above Franklin	104	2,000	13.6
Rabun Gap, head	118	2,200	14.3

NANTAHALA RIVER.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	1,550	.....
	6	1,700	25.0
	11	2,000	60.0
	17	2,500	83.3
	25	3,000	40.0
Head	40	4,200	80.0

TUCKASEEGEE RIVER.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	1,500	.....
Bryson	10	1,753	15.3
Dillsboro	27	2,000	14.5
Tuckaseegee	39	2,184	15.3
	48	2,700	57.3

FRENCH BROAD RIVER.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet*
Mouth, Knoxville	0	810	.....
	38	1,000	5.0
Delrio	49	1,100	9.1
Paint Rock	56	1,200	14.3
Barnard	76	1,500	15.0
Alexander	92	1,800	18.7
Above Asheville	111	2,000	10.5
Horseshoe	128	2,100	5.8
	160	2,200	3.1
Sassafras gap, head	169	2,800	66.6

BIG PIGEON RIVER.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	980	.....
	6	1,000	3.3
	16	1,100	6.2
State line, Tennessee and North Carolina	24	1,200	12.5
	26	1,300	50.0
	30	1,400	25.0
	34	1,500	25.0
	38	1,600	25.0
	43	1,700	20.0
	49	1,800	16.6
	50	1,900	100.0
	51	2,000	100.0
	54	2,100	33.3
	55	2,200	100.0
	56	2,300	100.0
	58	2,400	50.0
	69	2,500	9.1
	74	2,600	20.0
	80	2,700	16.6
Head	91	5,500	254.5

NOLICHUCKY RIVER.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	975	.....
	30	1,100	4.2
Bird bridge	45	1,200	6.6
Henshaw	53	1,300	12.5
	63	1,400	10.0
Near Conkling	75	1,500	8.3
	85	1,600	10.0
	90	1,700	20.0
	94	1,800	25.0
	96	1,900	50.0
	99	2,000	33.3
	101	2,100	50.0
	102	2,200	100.0
	104	2,300	50.0
Junction North and South Toe rivers	118	2,400	7.1
	123	2,500	20.0
	140	3,000	29.4
Head South Toe River	143	5,000	666.6

HOLSTON RIVER.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth, near Knoxville	0	810	.....
Near Morristown	78	900	1.2
Spears	105	1,000	3.7
	117	1,100	8.3
Rotherwood	143	1,156	2.2

SOUTH FORK OF HOLSTON RIVER.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	1,156	.....
	11	1,200	4.0
Mouth of Watauga River	16	1,300	20.0
Bluff	28	1,400	8.3
	41	1,500	7.7
Mouth of Fifteen-mile Creek	54	1,700	15.4
Mouth of Middle Fork	62	1,900	25.0
	74	2,100	16.6
	86	2,500	33.3

NORTH FORK OF HOLSTON RIVER.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth, Rotherwood	0	1,156	.....
	11	1,200	4.0
	21	1,300	10.0
	37	1,400	6.25
	50	1,500	7.7
	64	1,700	14.3
	82	2,000	16.6
Head	92	3,000	100.0

MIDDLE FORK OF HOLSTON RIVER.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	1,900	.....
Seven-mile ford	24	2,000	8.3
Head	44	2,500	25.0

POWELL RIVER.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	900	.....
	33	1,000	3.03
	61	1,100	3.6
Near Jonesville, Va	91	1,200	3.3
	103	1,300	8.3
Big Stone Gap	117	1,500	14.3
Near Norton	129	2,000	41.6

CLINCH RIVER.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth, Kingston	0	712	.....
Clinton	52	800	1.7
Near Mouth of Powell River	88	900	2.8
	116	1,000	3.6
Sneedville	128	1,100	8.3
	155	1,200	3.7
Dungannon	177	1,300	4.5
Near St. Paul	187	1,400	10.0
Artrip	207	1,500	5.0
	234	2,000	18.5
Near Tazewell	249	2,500	33.3

CUMBERLAND RIVER.

Cumberland River heads in the Cumberland Plateau in southeastern Kentucky and flows southwestward in a broad curve down into Ten-

nessee, and thence northwestward to its junction with the Ohio. The steep slope near its head is quickly reduced, and over much the greater part of its course its average descent is scarcely 1 foot to a mile, although it is interrupted in several places by slight rapids.

Its drainage basin comprises 18,573 square miles.

The profile is mainly from levels of the Engineer Corps, U. S. A.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	286	.....
C. O. & S. W. R. R. bridge	30	293	0.2
Glenwood landing	34	294	0.3
Eddyville	42	296	0.3
Commerce landing	49	299	0.4
Empire landing	54	302	0.6
Blight landing	58	304	0.5
Canton	61	306	0.7
Linton	72	309	0.3
Brandon landing	81	312	0.3
Carney landing	91	319	0.7
Cumberland City	104	325	0.5
Yellow Creek towhead	109	327	0.4
Palmyra	115	328	0.2
Clarksville	126	331	0.3
Seven-mile Island	132	334	0.5
Harrison landing	143	341	0.6
Harpeth shoals	156	352	0.8
Hickman ferry	166	363	1.1
Robertson Island	177	364	0.1
Nashville	189	366	0.2
Donelson ford	206	374	0.5
Lindsey Island	223	387	0.8
Gallatin	236	392	0.4
Cunningham Island	247	399	0.6
Buzzard Island	257	404	0.5
Whitley Island	265	410	0.8
Hartsville	276	415	0.5
Bradley Island	284	423	1.0
Lovell Island	294	432	0.9
Carthage	305	440	0.7
Beasley bar	318	448	0.6
Sullivan Island	323	450	0.4
Salt Lick Island	339	461	0.7

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Tackett Island	345	466	0.8
Simpson Island	353	475	1.1
Rose bar	364	478	0.3
Butler landing	373	489	1.2
Kentucky and Tennessee boundary	386	497	0.6
Sulphur Lick shoal	393	502	0.7
Stalcup Island	407	508	0.4
Cloyd Island	410	513	1.7
Green Island	432	527	0.6
Spearman Island	439	529	0.3
Wells Island	450	535	0.5
Blankenship Island	457	542	1.0
Long Bottom Island	461	546	1.0
Belks Island	472	554	0.7
Gauns Island	487	570	1.1
Ford Island	507	580	0.5
	513	585	0.8
Point Burnside	516	586	0.3
	548	700	3.6
	558	800	10.0
Williamsburg	587	900	3.4
Prineville	645	1,000	1.7
Forks, Mount Pleasant	678	1,100	3.0

KENTUCKY RIVER.

Kentucky River, like the Big Sandy, Guyandot, and Little Kanawha, heads in the Allegheny Plateau. Its head branches have steep slopes in the plateau, becoming more gentle in the Blue Grass country.

The profile is from levels by the Engineer Corps, U. S. A.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	410	.....
Frankfort	65	452	0.6
Beattyville	254	636	1.0
Mouth, Middle Fork	258	638	0.5

MIDDLE FORK OF KENTUCKY RIVER.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	638	.....
Mouth of War Creek	24	681	1.8
Troublesome Creek	54	731	1.7
Brashear's salt works	120	935	3.1

NORTH FORK OF KENTUCKY RIVER.

Locality.	Distance from Leatherwood.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth of Leatherwood Creek	0	935	.....
Mouth of Rockhouse Creek	9	994	6.6
Gum Spring	21	1,088	7.8
Whitesburg	27	1,141	8.8
Mouth of Boone Fork	38	1,252	10.1
Pound Gap	45	2,427	167.9

SOUTH FORK OF KENTUCKY RIVER.

Locality.	Distance from Leatherwood.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Beattyville	0	636	.....
Booneville	12	661	2.1
Red Bird	42	767	3.5
Collins Fork	69	842	2.8

WABASH RIVER.

This branch of Ohio River, having a drainage basin of 33,725 square miles, heads in northwestern Ohio and flows west and south to its mouth, after a course of 517 miles.

The profiles of Wabash and Eel rivers were compiled by Mr. Frank Leverett.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	311	.....
Mouth of Little Wabash	16	323	0.8
Grayville	62	365	0.9
Mouth of White River	90	377	0.4
Vincennes	122	399	0.7
Hutsonville	168	425	0.6
State line	197	441	0.6
Terre Haute	212	448	0.5
Covington	267	470	0.4
Attica	287	487	0.9
Lafayette	312	506	0.8
Logansport	362	583	1.5
Mouth of Mississinewa River	382	636	2.7
Mouth of Salamonie River	402	667	1.6
Huntington	417	699	2.1
Source	517	1,000	3.0

EEL RIVER.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth at Logansport	0	583	.....
Railroad bridge in Miami County	30	688	3.5
North Manchester	45	721	2.2
Liberty mills	50	750	5.8
Collamer	58	768	2.3
Columbia	70	816	4.0
Source	85	850	2.3

MUSKINGUM RIVER.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	570	.....
Foot of Zanesville dam	75	674	1.4
Dresden	91	700	1.6

MIAMI RIVER.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	427	.....
Middleton	55	656	4.2
Dayton	77	725	3.1
Piqua	108	840	3.7
Source	158	972	2.6

ILLINOIS RIVER.

Illinois River rises near the head of Lake Michigan and flows nearly southwest to its mouth, just above Alton. The slope of this river is very slight, being much less than that of the Mississippi above the mouth of the Illinois, its fall being only 110 feet in 324 miles, or about three-tenths of a foot per mile.

The profile is from levels by the Engineer Corps, U. S. A.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	405	.....
Peru, Ill	261	440	0.1
Little Rock ferry	265	446	1.5
Utica, Ill	266	448	2.0
Starved Rock	268	454	3.0
Buffalo Rock	271	459	1.7
Ottawa, Ill	277	463	0.7
Marseilles, Ill	284	475	1.7
Ballards Island	285	477	2.0
Seneca, Ill	289	484	1.8
S. & K. R. R. bridge	293	486	0.5
Morris, Ill	300	492	0.9
Aux Sable township	305	496	0.8
Kankakee feeder	310	501	1.0
Adams dam	324	515	1.0
Upper dam, Joliet	325	531	16.0

ROCK RIVER.

This stream heads in southern Wisconsin and flows south and southwest to its junction with the Mississippi at Rock Island. Although its profile is gentle throughout, it affords a number of fine water powers, several of which have been improved.

Its drainage basin is 9,792 square miles.

The profile is from the report on Water Power of the Tenth Census.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth at Rock Island, Ill	0	541	.....
Mouth of Green River	12	556	1.3
Mouth of Rock Creek	36	574	0.8
Mouth of Elkhorn Creek	65	601	0.9
Mouth of Leaf River	113	669	1.4
Mouth of Kishwaukee Creek	126	692	1.8
Mouth of Pecatonica River	151	716	1.0
Mouth of Catfish River	181	773	1.9
Fort Atkinson	199	784	0.6
Outlet Lake Horicon	261	861	1.2

RED RIVER.

The Red River of Texas has its source in the northern part of the Staked Plains. Its course for several hundred miles is a little south of east, along the northern boundary of Texas. At Fulton, Arkansas, it turns southeast and retains that general course to its mouth.

Throughout most of its course this river is graded, with a slope of only a few tenths of a foot to the mile, and along the south boundary of Indian Territory its slope is less than 2 feet per mile.

The drainage basin of Red River is 89,970 square miles, and its length considerably exceeds 1,000 miles.

The profile is derived in part from the levels of the Engineer Corps, U. S. A., and in part from levels by the United States Geological Survey.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	4	.....
Barbre landing	7	4	0.0
Black River	35	6	0.1
Ware landing	67	24	0.6
Cassandria	81	34	0.7
Casa landing	97	36	0.1
Alexandria	118	42	0.3
Colfax	152	52	0.3
Bell landing	165	58	0.5
St. Maurice	181	69	0.7
Tessier landing	188	72	0.4
Le Compte bluffs	211	86	0.6
Coushatta	237	96	0.4
Nicock bayou	273	125	0.8
Locust landing	305	131	0.2
Shreveport	327	138	0.3
Pandora bend	339	148	0.8
Cottonwood bayou	357	168	1.1
Elmer slough	375	179	0.6
Kouns canal	386	182	0.3
Blanton bluff	398	186	0.3
Collins bluff	407	188	0.2
Dukes bend landing	420	192	0.3
Booker landing	448	197	0.2
Garland	457	204	0.8
Person landing	468	208	0.4
Dobson landing	474	212	0.7
Dooley ferry	490	216	0.3
Kye Smith landing	504	221	0.4
Fulton, Ark	515	227	0.5
	705	500	1.4
	840	750	1.8
	955	1,000	2.2
	995	1,250	6.2
Western boundary of Oklahoma	1,045	1,500	5.0

OUACHITA RIVER.

The Ouachita River heads in the Ozark Hills of western Arkansas. In that part of its course which lies within the hills it flows in limestone

valleys between quartzite ridges, and has a considerable slope. Upon emerging into the level country near Arkadelphia, Arkansas, its slope greatly diminishes, and in its long course thence to its junction with Black River the fall per mile is very slight.

The profile is derived in part from levels by the Corps of Engineers, U. S. A., and in part from the atlas sheets of the United States Geological Survey.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	60	.....
Harrisonburg	16	67	0.4
Monroe	122	88	0.2
Mouth Bartholomew bayou	148	93	0.2
Camden	304	114	0.1
Arkadelphia	380	188	1.0
	417	300	3.0
	457	500	5.0
	476	600	5.3
	500	700	4.2
	514	800	7.1
	534	1,000	10.0
Head	545	1,750	68.2

ARKANSAS RIVER.

This large branch of the Mississippi heads in central Colorado, in the Rocky Mountains, at an altitude of 10,000 feet. It flows first south and then east, getting clear of the mountains at Canyon City, Colorado. Within the mountains its slope is extremely steep, averaging 40 feet to the mile; upon entering the plains its slope rapidly diminishes, and from Pueblo as far as the south boundary of Kansas, a distance of 500 miles, it remains almost constant, with an average slope of 7 feet per mile. In this part of its course it resembles the Platte. Although having a steep slope, it is so heavily loaded with detritus that it deposits rather than erodes. It is in effect a graded stream. From Wichita downward to its mouth its slope constantly diminishes, although not uniformly, and near its mouth it has no greater fall than the Mississippi in this neighborhood.

The drainage basin of the Arkansas is 185,671 square miles; the total length of the river is 1,497 miles.

The profile is derived as far as Argenta from levels of the Corps of Engineers, U. S. A.; thence to its head from atlas sheets of the United States Geological Survey.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	117	.....
Cut-off	15	118	0.1
Hopedale	22	119	0.1
Red Fork	27	121	0.4
Arkansas Post	38	130	0.8
Auburn	64	137	0.3
Little Bayou Meto	71	141	0.6
Sarassa	78	149	1.1
Bankhead	88	158	0.9
Rob Roy bridge	101	161	0.2
Pine Bluff	110	164	0.3
McAlister	121	170	0.5
Mokes landing	133	176	0.5
Red Bluff	142	188	1.3
Troy Landing	158	202	0.9
Argenta	176	216	0.8
	199	250	1.5
	249	300	1.0
	307	350	0.9
Fort Smith	403	400	0.5
Mouth of Grand River	497	500	1.1
Mouth of Cimarron River	584	652	1.7
Mouth of Black Bear Creek	646	764	1.8
Mouth of Salt Creek	661	797	2.2
Kaw Agency	723	933	2.2
Arkansas City	767	1,043	2.5
Oxford	792	1,108	2.6
El Paso	817	1,177	2.8
Wichita	832	1,222	3.0
Lyons	853	1,400	8.5
Hutchinson	871	1,500	5.6
Nickerson	889	1,600	5.6
Raymond	906	1,700	5.9
	926	1,800	5.0
	942	1,900	6.2

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Larned	958	2,000	6.2
	973	2,100	6.7
	989	2,200	6.2
	1,005	2,300	6.2
	1,020	2,400	6.7
	1,036	2,500	6.2
Cimarron	1,051	2,600	6.7
	1,066	2,700	6.7
Garden	1,081	2,800	6.7
	1,159	3,400	7.7
	1,172	3,500	7.7
Lamar	1,187	3,600	6.7
	1,200	3,700	7.7
	1,214	3,800	7.1
	1,229	3,900	6.7
Robinson	1,240	4,000	9.1
	1,254	4,100	7.1
	1,269	4,200	6.7
	1,283	4,300	7.1
	1,296	4,400	7.7
Mouth of Huerfano River	1,309	4,500	7.7
Mouth of St. Charles River	1,322	4,600	7.7
Pueblo	1,334	4,700	8.3
	1,345	4,800	9.1
	1,352	4,900	14.3
	1,359	5,000	14.3
	1,370	5,200	18.2
Canyon	1,375	5,300	20.0
Parkdale	1,385	5,700	40.0
	1,399	6,000	21.4
Mouth of South Arkansas River	1,428	6,500	17.2
	1,439	7,000	45.5
	1,457	8,000	55.6
Granite	1,475	9,000	55.6
Tennessee pass, head	1,497	10,400	63.6

CANADIAN RIVER.

This is a long branch of the Arkansas, heading in Raton pass, in northern New Mexico, and, flowing first south and then east down the slope of the plains, it joins the Arkansas in Indian Territory.

The profile is derived mainly from the atlas sheets of the United States Geological Survey.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	460	.....
	22	500	1.8
	82	600	1.7
	104	700	4.5
	134	800	3.3
	152	900	5.6
Near Purcell	190	1,040	3.7
C. R. I. & P. R. R. crossing	240	1,200	3.2
Canadian	435	2,300	5.6
Tascosa	550	3,150	7.4
	727	5,972	15.9
	745	6,292	17.8
Raton pass, head	758	7,893	123.2

NEOSHO RIVER.

Neosho River heads in Kansas, not far east of the center of the State, and flows at first eastward and then southward to its junction with the Arkansas in Indian Territory. Its course of 346 miles is mainly through a prairie region, and its profile departs but little from a normal one.

The profile is derived from the atlas sheets of the United States Geological Survey.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	470	.....
	12	500	2.5
Markham ferry, Ind. T	42	550	1.7
	69	600	1.8
Above Oswego, Kans	177	800	1.8
	206	850	1.7
Humboldt	228	900	2.3
Leroy	255	950	1.8
Ottumwa	274	1,000	2.6
Emporia	296	1,050	2.3
	310	1,100	3.6
Council Grove	326	1,200	6.25
Head	346	1,500	15.0

VERDIGRIS RIVER.

This river heads in southeastern Kansas and flows nearly south to its junction with the Neosho, in the northern part of Indian Territory. It drains a region mainly composed of prairie land. The river has slight fall except near its head, and with the exception of one point in its course it has a symmetrical profile.

The profile is derived from the atlas sheets of the United States Geological Survey.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	475	.....
	11	500	2.3
	60	550	1.02
	105	600	1.1
	133	650	1.8
	154	700	2.4
	179	750	2.0
	185	800	8.3
	206	850	2.4
	231	900	2.0
	242	950	4.5
	248	1,000	8.3
	258	1,100	10.0
	265	1,200	14.3
Head	275	1,500	30.0

WHITE RIVER.

This stream heads in northwestern Arkansas, in the Ozark Plateau, and flows southeastwardly through it in a canyon, with a steep slope. Emerging from the plateau, it flows southward through the alluvial region of southeastern Arkansas to its junction with Mississippi River, at the mouth of the Arkansas. Its slope throughout the alluvial region is extremely gentle. Its drainage basin is 27,925 square miles.

The profile is derived mainly from the atlas sheets of the United States Geological Survey.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	107	.....
Jacksonport	330	193	0.3
Grigsby ferry	380	250	1.1
	410	300	1.7
	439	350	1.7
	463	400	2.1
	487	450	2.1
	509	500	2.3
	536	550	1.9

ST. FRANCIS RIVER.

This stream heads in southeast Missouri and flows nearly south to its junction with the Mississippi, in eastern Arkansas. Nearly all its course is in low bottom lands of the Mississippi River, and its slope is extremely gentle.

The profile comes from levels by the Mississippi River Commission.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth, Ark	0	144	.....
Wittsburg, Ark	135	161	0.1
Ebdy, Butler County, Mo	308	314	0.9
Ironton, Mo	438	887	4.4

MERAMEC RIVER.

This is a small branch of the Mississippi, in eastern Missouri. It heads in the Ozark Plateau, and flows generally northeastward to its mouth, just below St. Louis.

The profile is derived from levels of the Engineer Corps, U. S. A.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	365	.....
Mouth of Big River	56	407	0.8
Mouth of Bourbeuse River	84	458	1.8
Mouth of Courtois River	134	615	3.1
Head	174	755	3.5

MISSOURI RIVER.

Missouri River has its source in southwest Montana, in three large branches—the Jefferson, Madison, and Gallatin—which meet at the Three Forks. These three streams have steep descents, mainly through mountainous regions, but from its head, at the Three Forks, the Missouri has a long course down the incline of the Great Plains, with a gentle slope, which is broken at only one point—Great Falls—by a succession of falls and rapids, in the course of which it descends nearly 200 feet. Below these falls it has the characteristics of a graded river, flowing through a broad bottom land, with wide curves and numerous shifts of its course. These characteristics become more and more marked in descending the river, accompanied by a gradually diminishing slope.

Its drainage basin, including all its branches, is 527,155 square miles.

The profile of Missouri River, from its mouth to the Three Forks, in Montana, is from levels of the Missouri River Commission. Those of the Jefferson, Madison, and Gallatin rivers are from atlas sheets of the United States Geological Survey.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	395	.....
Jamestown landing, Mo	9	403	0.9
St. Charles, Mo	28	416	0.7
Cottleville Landing, Mo	44	437	1.3
Washington, Mo	71	458	0.8
Hermann, Mo	103	479	0.7
Fishers landing, Mo	124	502	1.1
Jefferson City, Mo	151	523	0.8
Providence, Mo	180	545	0.8
Boonville, Mo	206	564	0.7
Glasgow, Mo	237	591	0.9
New Frankfort, Mo	252	602	0.8
Dewitt, Mo	267	614	0.8
Waverly, Mo	299	645	1.0
Lexington, Mo	322	664	0.8
Camden, Mo	337	678	0.9
Missouri City, Mo	363	695	0.7
Kansas City, Mo	391	716	0.8
Leavenworth, Kans	422	742	0.8
Atchison, Kans	448	765	0.9
St. Joseph, Mo	479	790	0.8
White Cloud, Kans	525	829	0.8
Brownsville, Nebr	578	875	0.9
Nebraska City, Nebr	608	908	1.1
Plattsmouth, Nebr	634	940	1.2
Omaha, Nebr	660	960	0.8
Blair, Nebr	695	986	0.7
Decatur, Nebr	745	1,033	0.9
Sioux City, Iowa	807	1,077	0.7
Vermilion, S. Dak	855	1,131	1.1
Mouth of James River, S. Dak	888	1,150	0.6
Yankton, S. Dak	898	1,161	1.1
Running Water, S. Dak	929	1,203	1.4
Fort Randall, S. Dak	969	1,236	0.8
Chamberlain, S. Dak	1,058	1,323	1.0

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth of Cheyenne River, S. Dak	1,112	1,460	2.5
Bismarck, N. Dak	1,239	1,618	1.2
Mouth of Little Missouri River	1,371	1,740	0.9
Williston, N. Dak	1,509	1,825	0.6
Mouth of Yellowstone River, N. Dak	1,549	1,855	0.8
Mouth of Poplar River, Mont	1,647	1,935	0.8
Mouth of Milk River, Mont	1,726	2,020	1.1
Mouth of Marias River, Mont	2,052	2,545	1.6
Fort Benton, Mont	2,074	2,565	0.9
Mouth of Portage River, Mont	2,099	2,783	8.7
Great Falls, Mo	2,111	3,295	42.7
Mouth of Sunrise River, Mo	2,123	3,299	0.3
Townsend, Mo	2,295	3,793	2.9
Three Forks, Mo	2,340	4,000	4.6

JEFFERSON RIVER.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Three Forks, mouth	0	4,000	--------
	24	4,200	8.3
	56	4,400	6.3
	66	4,600	20.0

BIGHOLE RIVER.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	4,600	--------
	16	4,800	12.5
	31	5,000	13.3
	43	5,200	16.7
	54	5,400	18.2
	64	5,600	20.0

BEAVERHEAD RIVER.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	4,600	.....
	19	4,800	10.5
	31	5,000	16.7
	51	5,200	10.0
	61	5,400	20.0

MADISON RIVER.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Three Forks, mouth	0	4,000	.....
	11	4,200	18.2
	21	4,400	20.0
	29	4,600	25.0
	40	4,800	18.2
	54	5,000	14.3
	63	5,200	22.2
	69	5,400	33.3
	79	5,600	20.0

GALLATIN RIVER.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Three Forks, mouth	0	4,000	.....
	12	4,200	16.7
	20	4,400	25.0
	24	4,600	50.0
	28	4,800	50.0
	34	5,000	33.3
	40	5,200	33.3
	44	5,400	50.0
	51	5,600	28.6
	59	5,800	25.0
	65	6,000	33.3
	69	6,200	50.0
	73	6,400	50.0
	77	6,600	50.0
	83	6,800	33.3
	88	7,000	40.0

OSAGE RIVER.

Osage River heads in eastern Kansas, under the name of Marais des Cygnes. Its slope throughout its course, of nearly 500 miles, is very gentle; indeed, for three-fourths of it it is less than a foot to a mile. This portion of Kansas is mainly through a prairie region, and here it is interrupted by marshes. In Missouri the river is extremely winding, in great curves, which are deeply incised.

The profile is derived from the atlas sheets of the United States Geological Survey.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth, Osage City	0	520	.....
Foot of Rice Island	12	524	0.3
Mouth of Big Gravois Creek	85	550	0.4
Near Crittenden	148	600	0.8
	216	650	0.7
Near Rockville	280	700	0.8
Near Pleasanton	350	750	0.7
	402	800	1.0
Ottawa	434	850	1.6
Near Melvern	486	950	1.9
Near Olivet	494	1,000	6.3

KANSAS RIVER.

The Kansas River and its branches, Smoky Hill and Republican rivers, are streams of the plains, heading in eastern Colorado and flowing down the long eastern slope of the country. Their courses are throughout gentle, although a little steeper toward the head, and less so in their lower courses.

Their drainage basins, comprising altogether 59,256 square miles, are treeless, except on the lower course of the main river.

These profiles are from the atlas sheets of the United States Geological Survey.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Kansas City, mouth	0	720	.....
Tiblow	20	754	1.7
Lawrence	50	796	1.4
Topeka	87	864	1.8
St. Mary	123	925	1.7
Wamego	139	959	2.1
St. George	150	977	1.6
Manhattan	160	996	1.9
Junction City	191	1,080	2.7
Abilene	254	1,100	0.3
Solomon	274	1,125	1.3
Salina	310	1,200	2.1
	353	1,300	2.3
	378	1,400	4.0
Ellsworth	408	1,500	3.3
	436	1,600	3.6
	463	1,700	3.7
	479	1,800	6.2
	496	1,900	5.9
	512	2,000	6.2
	528	2,100	6.2
	544	2,200	6.2
Wallace	654	3,280	9.8

REPUBLICAN RIVER.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth at Junction City	0	1,080	.....
Near Clay Center	38	1,200	3.2
Lawrenceburg	66	1,300	3.6
	86	1,400	5.0
	100	1,500	7.1
Near Bostwick	122	1.600	4.5
Near Red Cloud	143	1,700	4.8
Near Franklin	161	1,800	5.5
Republican	181	1,900	5.0
	199	2,000	5.5
Edson	215	2,100	6.3
	227	2,200	8.3

PLATTE RIVER.

Platte River heads in Colorado in two branches, North and South Platte. The former has its source in North Park and the mountains adjacent and has a steep descent within the mountains, dropping to 6 or 7 feet per mile when it enters the plains. The South Platte heads in the mountains at the north end of South Park and enters the plains just above Denver. Within the mountains its slope is extremely steep and irregular, but upon reaching the plains it suddenly diminishes greatly, falling to 8 or 9 feet to the mile. These two branches meet at North Platte, and below their junction the Platte has an average fall of about 6 feet per mile, maintaining that slope with remarkable uniformity. The river is a peculiar one in the fact that it has a relatively steep slope and an extremely straight course, while at the same time it is building up its bed. This peculiarity is due to the fact that it is, taking the year as a whole, an overloaded stream. It is subject to great fluctuations in volume. In the springtime, when the mountain snows are melting, it is a river a mile in width, and although rather shallow carries a large body of water, while at other times of the year it is almost or quite dry.

The drainage basin of Platte River comprises 90,011 square miles.

The profiles of the Platte and its branches are from the atlas sheets of the United States Geological Survey.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	940	.....
	15	1,000	4.0
	38	1,100	4.3
	63	1,200	4.0
	81	1,300	5.5
	101	1,400	5.0
	118	1,500	5.9
	133	1,600	6.7
Central City	149	1,700	6.3
	161	1,800	8.3
	176	1,900	6.7
	191	2,000	6.7
	206	2,100	6.7
Kearney	221	2,200	6.7
	237	2,300	6.3
Lexington	253	2,400	6.3
	270	2,500	5.9
	282	2,600	8.3
	300	2,700	5.6
North Platte	315	2,800	6.7

NORTH PLATTE RIVER.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth, near North Platte	0	2,800	.....
	15	2,900	6.7
Camp Clark	145	3,700	6.2
	161	3,800	6.3
	177	3,900	6.3
	193	4,000	6.3
	208	4,100	6.7
Fort Laramie	226	4,200	5.5
	239	4,300	7.7
	251	4,400	8.3
	265	4,500	7.1
	275	4,600	10.0
Douglas	310	4,827	6.5
Casper	370	5,100	4.6
Fort Steele	510	6,500	10.0

SOUTH PLATTE RIVER.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth, near North Platte	0	2,800	.....
Ogalalla	51	3,209	8.0
Big Spring	70	3,364	8.2
Denver Junction	81	3,456	8.4
Sedgwick	96	3,571	7.7
Crook	111	3,695	8.3
Iliff	127	3,820	7.8
Sterling	138	3,920	9.0
Merino	151	4,021	7.8
Snyder	168	4,160	8.2
Orchard	199	4,391	7.5
Hardin	216	4,513	7.2
Lasalle	232	4,663	9.4
Nantes	239	4,732	9.9
Platteville	244	4,807	15.0
Lupton	253	4,891	9.3
Brighton	260	4,968	11.0

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Henderson	265	5,023	11.0
Denver	288	5,170	6.4
Platte Canyon	308	5,492	16.1
South Platte	317	6,085	65.9
	332	6,500	27.7
	347	7,000	33.3
Mouth of Tarryall Creek	354	7,500	71.4
Foot of upper canyon	365	8,000	45.5
Head of upper canyon	375	8,165	16.5
	380	8,500	67.0
Mouth of Little Platte River	386	8,683	30.5
	405	9,000	16.7
Above Fairplay	415	9,500	50.0
	427	10,000	41.7

YELLOWSTONE RIVER.

Yellowstone River heads in the mountains above Yellowstone Lake, flowing into the lake at the head of its southeastern arm. On emerging from the lake the river has a very gentle slope for a few miles, then plunges over two falls, the upper fall 90 feet and the lower 325 feet. Thence follows a succession of canyons in volcanic rock, in which the river has a rapid descent. At Livingston it turns from its northern course to the east and flows through a broad valley with a diminishing slope to its mouth at Fort Buford.

The profile is in large part from the levels of the Northern Pacific Railway.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	1,855	.....
Diamond Island	30	1,916	2.0
Beef slough	46	1,950	2.1
Reno bend	60	1,988	2.7
Monroe rapids	100	2,100	2.8
Walker Island shoal	108	2,123	2.9
De Russy	113	2,133	2.0
White sand	124	2,160	2.5
McEwens rapids	133	2,182	2.4

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth of Powder River	137	2,200	4.5
Baker rapids	144	2,224	3.4
Buffalo rapids	163	2,300	4.0
Keogh, ferry at Tongue River	176	2,355	4.2
Benteen Island	196	2,402	2.3
Rosebud	215	2,465	3.3
Big Porcupine Creek	234	2,522	3.0
Head of Rosebud Island	252	2,583	3.4
Mouth of Bighorn River	274	2,668	3.9
Junction City	279	2,696	5.6
N. P. R. R. crossing, near Billings	331	3,079	7.4
Stillwater	373	3,560	11.5
W. P. R. R., second crossing	385	3,676	9.7
Gray Cliff	400	3,847	11.4
Big Timber	413	4,072	17.3
Springdale	427	4,190	8.4
Livingston	446	4,437	13.1
Head of lower canyon	456	4,600	16.3
Chicory	468	4,800	16.7
Foot second canyon	486	5,000	11.1
	533	6,000	21.3
Foot of lower falls	552	7,300	68.4
Top of upper falls	553	7,725	425.0
Yellowstone Lake	569	7,741	1.0

MILK RIVER.

This is a long branch of Missouri River, in northern Montana. It heads in the plains near the international boundary, and flows in a course generally a little south of east, to its mouth. Its slope conforms to that of the plains, averaging about 2 feet per mile.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	2,020	.....
Glasgow, 2 miles above	22	2,055	1.6
Malta	106	2,220	2.0
Yantic	184	2,415	2.5

JAMES RIVER.

James River, of the Dakotas, is a stream of the plains, flowing through its entire course in a region recently occupied by the great northern glacier. It heads in North Dakota and flows nearly south 433 miles, to its junction with Missouri River at Yankton. Its entire course is newly occupied and the river has made but little progress in erosion. It follows closely in its profile the slopes of the country, which are very gentle.

The profile is derived from the atlas sheets of the United States Geological Survey.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	1,150	.....
Olivet, 4 miles above	42	1,180	0.7
	64	1,200	0.9
	105	1,220	0.5
	172	1,240	0.3
	211	1,260	0.5
	265	1,280	0.4
Grand Rapids	315	1,300	0.4
Dickey, 2 miles above	326	1,320	1.8
	334	1,340	2.5
	348	1,360	1.4
Jamestown, 3 miles below	358	1,380	2.0
Jim Lake	382	1,435	2.3
Arrowhead Lake	394	1,440	0.4
New Rockford	433	1,502	1.6

DES MOINES RIVER.

This river heads in southwest Minnesota and flows in a southeast course across Iowa to its junction with the Mississippi at Keokuk. Its slope is gentle throughout its course and fairly uniform.

Its drainage basin is 14,652 square miles.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth, Keokuk, Iowa	0	476	.....
Ottumwa, Iowa	94	636	1.7
Des Moines, Iowa	205	786	1.4
Moingona, Iowa	245	877	2.3
Southeast part of Webster County, Iowa	269	921	1.8
Fort Dodge, Iowa	300	964	1.4
Windom, Minn	411	1,329	3.3

IOWA RIVER.

This branch of the Mississippi heads in northern Iowa and flows southeast to its mouth. Its course is through a prairie region and its slope is gentle throughout.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	522	.....
Iowa City	60	607	1.4
Montour	150	845	2.6
Iowa Falls	215	1,007	2.5

SKUNK RIVER.

A branch of Mississippi River in eastern Iowa. This stream shows, so far as the data at hand indicates, a very regular profile.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	505	.....
Rome	38	550	1.2
Vowell	146	750	1.9
Ames	182	907	4.4
Southeast part of Hamilton County	203	1,056	7.1

MINNESOTA RIVER.

This river heads in Bigstone Lake, on the boundary between Minnesota and South Dakota. After a long course to the southeast it turns sharply northward and joins the Mississippi near Minneapolis. Its slope throughout is extremely gentle, ranging from one-tenth of a foot per mile to two and eight-tenths, and with an average slope for its entire length of 249 miles of only one and one-tenth feet per mile. Throughout its course the river flows in a bottom land of considerable breadth, entirely out of proportion to the size of the present stream, indicating that in former times a much larger river excavated its present valley. Its drainage basin is 16,000 square miles.

The profile is from levels by the Corps of Engineers, U. S. A.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	688	.....
Shakopee	26	689	.....
Carver	33	691	0.3
Jordan ferry	41	693	0.3
Belleplaine	52	697	0.4
Blakeley	60	701	0.5
Henderson	73	711	0.8
Lesueur	82	716	0.6
Ottawa	90	723	0.9
St. Peter	97	731	1.1
Mankato	115	753	1.2
South Bend	119	758	1.3
Fort Ridgely	131	791	2.8
Mouth of Redwood River	146	815	1.7
Mouth of Yellow Medicine River	173	859	1.6
Mouth of Chippewa River	197	923	2.7
Lac qui Parle	211	938	1.1
Mouth of Pomme de Terre Creek	227	946	0.5
Bigstone Lake	249	976	1.4

RED RIVER.

Red River, of Minnesota, heads in Lake Traverse, and forms the boundary line between Minnesota on the east and the Dakotas on the west as far as the international boundary. Its course from its head to its mouth in Lake Winnipeg is very nearly north.

The profile is from levels by the Corps of Engineers, U. S. A.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth, Lake Winnipeg	0	710	.....
Boundary line	120	753	0.4
Pelican	168	762	0.2
Turtle River	238	783	0.3
Grand Forks	263	789	0.2
Frog Point	298	811	0.6
Goose Rapids	320	823	0.5
Moorhead	418	874	0.5
Fort Abercrombie	489	912	0.5
Breckenridge	515	948	1.4
Fergus Falls	548	1,156	6.3

COLORADO RIVER.

Colorado River drains a peculiar region. While most of its water comes from high mountain ranges, the greater part of its course, as well as the courses of its tributaries, lies in the plateau region, an area of horizontal or slightly inclined plateaus, bordered by cliffs, in which the streams flow in steep-walled canyons; an area sparsely covered with soil and containing little vegetation. It is a region of slight rainfall, and the streams lose rather than gain in volume of water in traversing it. Colorado River heads in two main branches—Green and Grand rivers. The sources of Green River are in the Wind River Mountains of western Wyoming. At the foot of this range the river traverses for a hundred miles a broad, desert plain, known as Green River Basin. At the foot of this plain it meets Uinta Range. Through this range it has cut its way in a series of heavy gorges, emerging from them at the south base of the mountains and entering Uinta Valley, in which it flows for a short distance; then flowing southward it enters a series of uplifts, consisting of slightly inclined plateaus, three in number, dipping to the north. In each of these it burrows its way, the depth of the canyon increasing mile by mile, both by the increasing height of the plateau and the descent of the river. At the cliff which limits each of these plateaus on the south the river comes out to daylight for a short distance. At the foot of the most southern of these plateaus is Gunnison Valley, in which the Rio Grande Western Railroad crosses the river. In the canyon which succeeds this valley, caused by an uplift of the plateau, the Green is joined by the Grand, and the Colorado begins. Then follows a succession of uplifts, through which the river has been forced to cut its way, producing a continuous canyon, whose walls rise higher and higher with each succeeding uplift until the Grand Canyon is reached, whose walls are 6,000 feet in height and magnificent in their complexity. Farther west the land descends by a series of steps, produced by faults and folds, which finally bring the river to daylight at the mouth of the Grand Wash.

Grand River, which heads in the mountains in the eastern side of Middle Park, Colorado, encounters throughout its course a succession of obstacles similar to those met and overcome by Green River.

The branches of Grand River, Eagle River, Roaring Fork, Gunnison and Dolores rivers all head and have much of their courses in regions of high mountains, and naturally their slopes are both steep and irregular.

The entire area drained by Colorado River and its branches is 225,049 square miles.

The profile was prepared by Maj. J. W. Powell.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	0	.....
Mouth of Gila River	150	125	0.8
Mouth of Williams River	340	375	1.3
Needles	385	448	1.6
Mouth of Virgin River	555	935	2.9
Mouth of Grand Wash (fault)	600	1,000	1.4
Mouth of Diamond Creek	650	1,312	6.2
Toroweap Valley (fault)	700	1,625	6.3
Mouth of Kanab Creek	730	1,810	6.2
	790	2,300	8.2
	800	2,520	22.0
Mouth of Little Colorado River	815	2,625	7.0
Mouth of Paria River (fault—Echo Cliffs)	880	3,187	8.6
Mouth of Navajo Creek	905	3,220	1.3
Crossing of the Fathers	920	3,250	2.0
Mouth of San Juan River	957	3,310	1.6
Mouth of Escalante River	970	3,325	1.2
Mouth of Dirty Devil River	1,030	3,434	1.8
	1,067	3,750	31.2
Mouth of Grand River	1,080	3,775	1.9
Green River crossing (Book Cliffs)	1,200	4,075	2.5
Mouth of Price River	1,220	4,200	6.25
	1,305	4,575	4.4
Mouth of White River	1,390	4,625	0.6
	1,435	4,750	2.8
Mouth of Yampa River	1,470	5,100	10.0
Brown park	1,490	5,375	13.8
Mouth of Henry fork	1,560	5,813	6.3
Mouth of Black fork	1,590	5,940	4.2
Green River city	1,620	6,075	4.5
Mouth of Big Sandy River	1,652	6,240	5.2
Mouth of Slate Creek	1,672	6,500	13.0
Mouth of Fontenelle Creek	1,684	6,620	10.0
Mouth of New Fork	1,728	6,900	6.3
Mouth of Horse Creek	1,754	7,180	10.8
Mouth of Lead Creek	1,769	7,383	13.5
	1,789	7,622	11.95
Bend, near head	1,800	7,808	16.9

LITTLE COLORADO RIVER.

This stream heads in the southern edge of the Mogollon Plateau, in eastern Arizona, and flows nearly northwest to its junction with the Colorado. Near the head its slope is steep, but through most of its course across the plateau, on the surface of which it flows, its slope is gentle. Toward its mouth it begins to burrow into the plateau, and its slope increases, finally, just before reaching the Colorado, becoming very steep, producing a curve near its mouth convex upward.

The profile is derived from the atlas sheets of the United States Geological Survey.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	2,625	.....
	6	2,750	20.8
	16	3,000	25.0
	24	3,250	31.3
	32	3,500	31.3
	40	3,750	31.3
	50	4,000	25.0
	66	4,250	15.6
	84	4,500	13.9
	126	4,750	5.9
	158	5,000	7.8
	188	5,200	6.7
	204	5,400	1.3
Near St. Johns	221	5,600	11.8
	235	5,800	14.3
	244	6,000	22.2
	250	6,200	33.3
	252	6,400	100.0
	254	6,600	100.0
	256	6,800	100.0
	262	7,000	33.3
Head	277	8,000	66.7

SAN JUAN RIVER.

This branch of Colorado River heads in many streams flowing south from the San Juan Mountains—Mancos, La Plata, Los Piños, and Animas rivers. At their south base it collects the waters of these several branches and thenceforward flows westward through a plateau region to its junction with the Colorado. In the mountains these streams

have steep descents, which are suddenly checked on reaching the plateau. In its course down the plateau, however, the slope of San Juan River is by no means gentle or regular. A feature of the slope of this river is the increase in its grade toward its mouth, showing that the parent stream, the Colorado, has thus far been able to cut its grade down more rapidly than the smaller stream.

The profiles of this river and its branches have been derived mainly from the reports of the Hayden survey.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	3,310	.....
	16	3,500	11.8
	58	3,750	5.9
	80	4,000	11.4
Bluff City	98	4,250	13.9
Four miles below mouth of Montezuma Creek	107	4,390	15.6
Mouth of McElmo Creek	118	4,510	10.9
	130	4,540	2.5
Mouth of Mancos River	143	4,700	12.3
	164	4,880	8.6
Pictured Rocks	186	5,180	13.6
Mouth of La Plata River	197	5,297	10.6
Mouth of Animas River	200	5,310	4.3
Mouth of Los Pinos River	240	5,750	11.0
Mouth of Piedra River	265	6,000	10.0
Pagosa Springs	303	7,095	28.8

RIO MANCOS, COLORADO.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	4,700	.....
	12	4,900	16.7
	22	5,270	37.0
	32	5,730	46.0
	38	6,250	86.7
Merritt ranch	52	7,360	79.3
	62	9,770	241.0

LA PLATA RIVER, COLORADO.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	5,297	.....
	10	5,500	30.3
	22	6,270	64.2
	38	7,922	103.3
Parrott City	43	8,500	115.6

ANIMAS RIVER.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	5,310	.....
Mouth of Florida River	38	6,106	20.9
Foot of Animas park	56	6,600	27.4
Head of Animas park	70	6,900	21.4
Mouth of Cascade Creek	80	7,700	80.0
Foot of Bakers park	97	9,400	100.0
Head of Bakers park	105	9,900	62.5
Divide at head	113	12,500	325.0

LOS PINOS RIVER, COLORADO.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	5,750	.....
Trail crosses	33	6,680	28.2
Big bend	42	7,288	67.6
Mouth of Vallecito Creek	48	7,688	66.7
Mouth of West Branch	60	8,688	83.3
Trail leaves stream	69	9,888	133.3
Weeminuche pass	75	10,670	130.3

GRAND RIVER, COLORADO.

[The profiles of this river and its branches are from the reports of the Hayden survey.]

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	3,775	.....
Mouth of Dolores River	52	4,250	9.1
Horseshoe bend	70	4,300	2.8
Head of low canyon	104	4,500	5.9
Mouth of Gunnison River	120	4,523	1.4
Mouth of Roan Creek	152	5,100	18.0
Mouth of North Mam Creek	188	5,645	15.1
Mouth of Roaring Fork	209	5,743	4.7
Mouth of Eagle River	228	6,125	20.1
Foot of canyon in Park Range	295	7,000	13.1
Mouth of Muddy Creek	302	7,180	25.7
Hot Springs	320	7,715	29.7
Forks	344	8,123	17.0
Grand Lake, Middle Park	348	8,153	7.5

RIO DOLORES, COLORADO.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	4,250	.....
Mouth of Unaweep Canyon	21	4,600	16.7
Mouth of San Miguel River	43	5,000	18.2
In Paradox Valley	49	5,100	16.7
Mouth of Disappointment Creek	83	6,500	41.2
Mouth of Lost Canyon	134	6,950	8.8

GUNNISON RIVER.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	4,523	.....
Mouth of Roubideau Creek	40	4,925	10.1
Mouth of Uncompahgre River	45	5,100	35.0
Mouth of North Fork	62	5,405	17.9
Mouth of Cebolla Creek	97	6,800	39.8
Mouth of Lake Fork	112	7,213	27.5
Mouth of White Earth River	123	7,450	21.5
Foot of open valley	130	7,638	26.9
Mouth of Tomichi Creek	141	7,725	7.9
Mouth of Slate River	157	8,176	28.2
Head of upper canyon	176	9,576	73.7
Mouth of Pass Creek	185	9,865	32.1
Head	200	11,000	75.7

UNCOMPAHGRE RIVER.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0.0	5,100	.....
Ford of Salt Lake road	29.5	5,800	23.7
Uncompahgre Agency	40.0	6,400	57.1
Mouth of Dallas Fork	54.5	7,000	41.4
Lower end of canyon	68.5	8,000	71.4
Lower end of small valley	72.5	9,500	375.0
Head of small valley	74.5	9,700	100.0
Divide at head	78.5	11,100	350.0

LAKE FORK OF GUNNISON.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0.0	7,213	.....
Mouth of Indian Creek	15.0	7,860	43.1
Mouth of Goodwin Creek	36.0	8,660	38.1
Mouth of South Branch	51.0	9,860	80.0
Valley	56.5	11,060	218.2
Head	59.0	13,260	880.0

ROARING FORK.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	5,743	.....
Mouth of Rock Creek	12	6,000	21.4
Mouth of Frying-pan Creek	25	6,626	48.2
Mouth of Castle Creek	43	7,942	73.1
Mouth of Difficult Creek	48	8,241	59.8
Mouth of Hunter Creek	55	9,400	165.6
Head	64	11,676	252.9

EAGLE RIVER, COLORADO.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	6,125	.....
Head of canyon	29	7,065	32.4
Mouth of Gore Creek	41	7,700	52.9
Mouth of Roche Moutonnée Creek	45	7,856	39.0
Mouth of Homestake Creek	50	8,693	167.4
Head in Tennessee pass	62	10,418	143.7

SEVIER RIVER.

Sevier River heads in the plateaus in southern Utah, and flows for a long distance northward through valleys separating plateaus and mountain ranges, with a steep but gradually diminishing slope. In central Utah it turns sharply, flowing west in a canyon through the Canyon Range, where its descent is somewhat steeper, and out into Sevier Desert, through which it flows with a very gentle slope to Sevier Lake, into which it sinks.

The profile is derived from the atlas sheets of the United States Geological Survey.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Sevier Lake, mouth	0	4,600	.....
Near Deseret	34	4,638	1.1
In canyon	88	4,750	2.1
Above Salina	152	5,000	3.9
Joseph	180	5,250	8.9
	189	5,500	27.8
	205	5,750	15.6
Circle Valley	223	6,000	13.9
	243	6,250	12.5
Near Panguitch	255	6,500	20.8
Near Hillsdale	265	6,750	25.0
	271	7,000	41.7
Head	279	7,500	62.5

BEAR RIVER.

Bear River heads in the northern slope of Uinta Mountains in northeastern Utah. The steep slope of the mountain side is succeeded by a comparatively gentle slope as it flows northward through a succession of broad valleys. At Soda Springs it turns sharply upon itself and flows nearly south, and immediately descends from the surface of a basalt flow to its base, producing a succession of falls and rapids. Through Cache Valley its slope is very gentle, but at The Gates, where it emerges from Cache Valley into the valley of Great Salt Lake, it has a steep descent in a series of rapids.

The profile is from the measurements of the Hayden survey.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	4,218	.....
Hampton Bridge	24	4,353	5.6
Head of Gates	30	4,450	16.2
Mouth of Battle Creek	66	4,499	1.4
Foot of canyon below Gentile Valley	73	4,609	15.7
Head of canyon below Gentile Valley	88	4,692	5.5
Bend below Sheep Rock	113	5,737	41.8
Soda Springs	125	5,855	9.8
	151	5,900	1.7
Foot of canyon above Bear Lake Valley	171	5,989	4.5
Mouth of Smith Fork	201	6,223	7.8
	216	6,254	2.1
	243	6,353	3.7
Bend	251	6,505	19.0
Evanston	270	6,800	15.5
Head	320	11,000	84.0

HUMBOLDT RIVER.

This river of Nevada heads in the northeastern part of the State and flows southwestward, sinking in Humboldt Lake, in the western part of the State. In its course of more than 300 miles it flows almost directly across the series of uplifts forming the Basin ranges, but in most cases it passes through broad gaps in these ranges rather than in canyons. The drainage basin is 32,148 square miles.

The profile is from the Fortieth Parallel Survey.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Humboldt Lake, sink	0	3,900	.....
Lovelock	8	3,950	6.3
Oreana	20	4,150	16.7
Humboldt	40	4,200	2.5
Raspberry	68	4,300	3.6
Winnemucca	90	4,325	1.1
Golconda	108	4,350	1.4
Stonehouse	133	4,400	2.0
Battle Mountain	153	4,500	5.0
Shoshone	178	4,600	4.0

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Palisade	203	4,800	8.0
Moleen	223	4,975	8.8
Osino	244	5,100	5.9
Halleck	258	5,200	7.1
Deeth	272	5,300	7.1
Humboldt Wells	292	5,600	15.0
Independence, head	308	6,000	25.0

SACRAMENTO RIVER.

Sacramento River heads in the mountains of northern California, principally in the easternmost of the coast ranges. Its head streams, upon reaching the great depression between the Sierra Nevada and the Coast Ranges, turns south and by a steep descent in a deep canyon it flows off a volcanic plateau which surrounds Mount Shasta. Having reached the level of the great California Valley, its slope rapidly diminishes, and below Red Bluff becomes very slight. Indeed, from Sacramento to the bay the descent is trifling, the river meandering most of the way through great tule marshes.

This river has been in times past overloaded with detritus from placer mines, which have caused it to deposit freely, building up its course above the adjacent country. Much of the city of Sacramento lies at present below the level of the river when in flood and is protected from it by levees.

The profile is from the levels of the Engineer Corps, U. S. A.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
New York landing	0	0	.....
Collinsville	3	0	.....
Rio Vista	17	1	.....
Head of Grand Island	37	5	0.1
Heacock shoals	54	7	0.1
Sacramento	64	9	0.2
Mouth of Feather River	84	16	0.35
Knight landing	99	20	0.3
Winn landing	120	27	0.3
Colusa	155	43	0.5
Caldins	167	57	1.2
John Boggs landing	171	62	1.3

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Princeton	175	67	1.3
Butte	180	73	1.2
Jacinto	192	93	1.7
Parrott landing	195	98	1.66
Monroeville	203	110	1.5
Chico landing	209	119	1.5
Bidwell landing	214	128	1.8
Gazelle chute	230	157	1.8
Squaw Hill	236	168	1.8
Tehama	247	201	3.0
Sacramento bar	255	225	3.0
Last Chance	261	237	2.0
Red Bluff	265	245	2.0
	306	400	3.8
Near Buckeye	326	600	10.0
	344	800	11.1
Delta	356	1,000	16.7
	364	1,200	25.0
Southern	368	1,600	100.0
Upper Soda Spring	379	2,000	36.4
	384	3,000	200.0
Head	399	7,000	266.7

PIT RIVER.

[From the atlas sheets of the United States Geological Survey.]

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	687	.....
	13	1,000	24.1
	17	1,200	50.0
	31	1,400	14.3
	40	1,600	22.2
	47	1,800	28.6
	51	2,000	50.0
Near Fall River mills	75	3,000	41.7
Above Pittville	86	3,400	36.4
Foot of Big Valley	107	4,000	28.6
	127	4,200	10.0
Canby	152	4,350	6.0
Alturas Hill	176	4,446	4.0
Goose Lake	196	4,800	17.7

OTHER CALIFORNIA RIVERS.

The Sierra streams, Feather, American, Yuba, Mokelumne, Calaveras, Stanislaus, Tuolumne, and Cosumnes rivers, are all characterized by extremely steep and irregular slopes.

Pit River is one of the largest branches of the Sacramento. It heads in Goose Lake, in northeastern California. For some distance below the lake it has a comparatively gentle course. It cuts its way through the Sierra Nevada in a deep gorge, in the upper part of which the slope is gentle, but after passing the divide of the range its course becomes extremely steep, and this slope is maintained with but little diminution to its mouth.

FEATHER RIVER.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	0	.....
Nicolaus	10	9	0.9
Yuba River	20	21	1.2
Burt ferry	44	56	1.5
Oroville	57	100	3.4
Middle Fork	64	198	14.0
West Branch	73	648	50.0
Upper end Big Bend	85	1,080	36.0
Buck Creek	104	1,954	46.0
Chip Creek	112	2,370	52.0
East Branch	115	2,559	63.0
Carriboo	121	2,949	65.0
Bidwell bridge	136	4,678	115.3

YUBA RIVER.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth, Marysville	0	60	.....
Long bar	15	100	2.6
Forks, North and South	28	500	30.8
Forks of North Fork	36	1,000	62.5
Up North Fork, Slate Range bar	53	2,000	58.8
Downieville	70	3,000	58.8
Loganville	78	4,000	125.0
Tehuantepec Valley	90	6,700	225.0

AMERICAN RIVER.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth at Sacramento	0	10	.....
	23	100	3.9
Forks	28	200	20.0
Up North Fork	37	300	11.1
	41	400	25.0
	44	500	33.3
Mouth Middle Fork	48	600	25.0
	56	700	12.5
	59	800	33.3
Toll House	66	1,000	28.6
	77	1,500	45.5
Mouth of North Fork	83	2,000	83.3
	94	3,000	90.9
	100	4,000	166.7
	105	5,000	200.0
	112	6,000	142.8
	118	8,500	416.7

COSUMNES RIVER.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth, Benson bridge	0	25	.....
	25	100	3.0
	34	200	11.1
	44	500	30.0
Forks, North and South	47	800	100.0
Up North Fork	54	1,000	28.6
Sweeney	66	2,000	83.3
Dyer mill	72	3,000	166.7
	78	4,000	166.7
	83	5,000	200.0
	89	6,000	166.7
Head	93	7,500	375.0

MOKELUMNE RIVER.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Junction with Cosumnes River	0	25	.....
	28	100	2.7
	39	200	9.1
	47	500	37.5
	58	1,000	45.5
	66	2,000	125.0
	78	3,000	83.3
	90	4,000	83.3
	98	5,000	125.0
	103	6,000	200.0
Hermit Valley	109	7,000	166.7
Head	116	8,000	142.9

CALAVERAS RIVER.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	5	.....
Bellota	23	100	4.1
	44	200	4.8
	51	500	42.9
	68	1,000	29.4

STANISLAUS RIVER.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	30	.....
Oakdale	26	200	6.5
Parrott ferry	65	1,000	20.5
Forks	75	1,300	30.0
	79	2,000	175.0
	83	3,000	250.0
	89	4,000	166.7
	97	5,000	125.0
	102	6,000	200.0
	113	8,000	181.8

TUOLUMNE RIVER.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	40	.....
Lagrange	50	300	5.2
	59	500	22.2
	74	1,000	33.3
Forks	79	1,200	40.0
	87	1,600	50.0
	98	2,000	36.4
Canyon	109	3,000	90.9
	125	4,000	62.5
	133	5,000	125.0
	137	6,000	250.0
	138	7,000	1,000.0
Head of canyon	141	8,000	333.0
Meadows	144	8,500	167.0
	154	9,000	50.0
	155	10,000	1,000.0

KLAMATH RIVER.

Klamath River heads in Klamath Lake, in southern Oregon, and flows westward through the Coast Mountains by a somewhat sinuous course to the Pacific. It has a rapid and an extremely irregular fall, showing that its existence under the present topographic conditions has been brief. Its drainage basin is 14,660 square miles.

The profile is from the atlas sheets of the United States Geological Survey.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	0	.....
	127	1,800	14.2
	146	2,000	10.5
	158	2,200	16.7
	159	2,400	200.0
	161	2,600	100.0
	177	2,800	12.5
	185	3,800	125.0
	194	4,000	22.2
Linkville, outlet Klamath Lake	214	4,186	9.3

COLUMBIA RIVER.

Columbia River, including Clark Fork, heads in the Rocky Mountains, west of Helena, Montana. Its slope is great from its head to the crossing of the Northern Pacific Railroad, below the mouth of Flathead River. In northern Idaho it flows through Lake Pend Oreille, and below that, in its northward course to the international boundary, it has a steeper descent. In this part of its course it is in a deep canyon. Thenceforward for several hundred miles it has a gentle slope, interrupted by rapids of no great magnitude until Kettle falls are reached. Other rapids succeed, the most formidable being Rock Island rapids, which form a complete obstruction to navigation. In its lower course in passing the Cascade Range it encounters two notable rapids, the Cascades and the Dalles, both of which interrupt navigation.

The entire drainage area of Columbia River is 216,537 square miles, about half of which belongs to Columbia River proper, and half to its main branch, Snake River.

The profile of the Columbia is in part from the work of Lieutenant Symons, in part from the atlas sheets of the United States Geological Survey.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	0	.....
Mouth of Snake River	312	145	0.5
	331	400	13.4
Rock Island rapids	466	594	1.4
Wenatchee	473	618	3.4
Orondo	494	665	2.2
Troy	504	680	1.5
Chelan falls	515	700	1.8
Mouth of Methow River	536	750	2.4
Virginia City	542	760	1.7
Mouth of Okanogan River	546	770	2.5
Mouth of Spokane River	655	1,073	2.8
Below Grand rapids	692	1,167	2.5
Below Kettle falls	699	1,191	3.4
Above Kettle falls	699	1,224	.....
North boundary	740	1,305	2.0
Pend Oreille Lake, foot	854	2,062	6.6
Pend Oreille Lake, head	881	2,062	.....
Crossing N. P. R. R	930	2,298	4.8
Missoula	1,072	3,191	17.8
Near Bonita	1,097	3,621	17.2

WILLAMETTE RIVER.

This river, one of the main southern branches of the Columbia in Oregon, heads in the Cascade Range. Flowing down its slopes to the center of the Willamette Valley, it turns northward and pursues this course to its junction with the Columbia, just below the city of Portland. The profile represents only a portion of its course in the Willamette Valley. It shows a fairly uniform slope, gradually diminishing downward until the falls at Oregon City are reached, where there is an abrupt drop of 41 feet, with a second drop just below it, at Clackamas rapids, amounting to 5½ feet.

The drainage basin of the Willamette River is 11,700 square miles.

The profile is from the levels of the Engineer Corps, U. S. A.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	0	.....
Portland	12	0	.....
Foot of Clackamas rapids	23	0.5	.....
Head of Clackamas rapids	24	6	5.5
Oregon City, below falls	26	6	.....
Oregon City, above falls	26	47	.....
Rogers landing	49	50	0.1
Salem	84	107	1.6
Independence	95	129	2.0
Albany	119	165	1.5
Corvallis	131	184	1.6
Peoria	144	216	2.5
Harrisburg	164	286	3.5
Eugene	184	396	5.5

FLATHEAD RIVER.

Flathead River heads in Canada and flows southward, first with a steep and then with a gentle slope, into Flathead Lake. Below this lake it turns to the west and joins the Columbia.

Its profile is in part from the levels of the Northern Pacific Railway, in part from the atlas sheets of the United States Geological Survey.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	2,410	.....
Crossing N. P. R. R.	15	2,464	3.6
Flathead Lake, foot	81	2,874	6.2
Flathead Lake, head	101	2,874	.....
	151	3,000	2.5
	168	3,400	23.5
Near north boundary	186	3,800	22.2

SNAKE RIVER.

Snake River heads in Shoshone Lake in Yellowstone Park, thence it flows through Lewis Lake and, at the head of Jackson Hole, through Jackson Lake. Below the latter lake it has a steep descent, which is greatly increased in the canyon through which it passes from Jackson Hole to Snake River plains. Here it flows on the surface of a basalt field, and its course for some distance is extremely sluggish, through a broad extent of marshes. It soon, however, begins to cut into the basalt and to quicken its course to such an extent that it was in the early days called Mad River.

In descending from bench to bench in this basalt field, produced by successive flows of lava, it has falls, American falls near the mouth of Portneuf River, and Shoshone falls, lower down. Below Shoshone falls its slope is quite gentle for a long distance, but increases greatly in the canyon by which it cuts through the Blue Mountains. Below this canyon the slope to its mouth is very gentle. It is navigable to Lewiston, Idaho.

It is a large stream, having a total length of 939 miles and a drainage area of 103,835 square miles, or nearly half that of Columbia River.

The profile is almost entirely from the atlas sheets of the United States Geological Survey.

Locality.	Distance from mouth.	Height above sea.	Fall per mile.
	Miles.	*Feet.*	*Feet.*
Mouth	0	145	.....
Weiser	306	2,123	6.5
Mouth of Malheur and Payette rivers	324	2,152	1.6
	363	2,200	1.2
	391	2,230	1.1
	417	2,300	2.7
	430	2,350	3.8
Glenns Ferry	480	2,500	3.0
	602	4,190	13.9
	620	4,205	0.8
Mouth of Rock Creek	644	4,242	1.5
Mouth of Portneuf River	669	4,335	3.7
Anderson	682	4,362	2.1
	750	5,030	9.8
Mouth of Salt River	807	5,363	5.8
In canyon	823	5,909	34.1
Mouth of Gros Ventre River	855	6,227	9.9
Mouth of Lake Creek	867	6,440	17.7
Foot of Jackson Lake	887	6,808	18.4
Lewis Lake	931	7,720	20.7
Shoshone Lake	939	7,746	2.5

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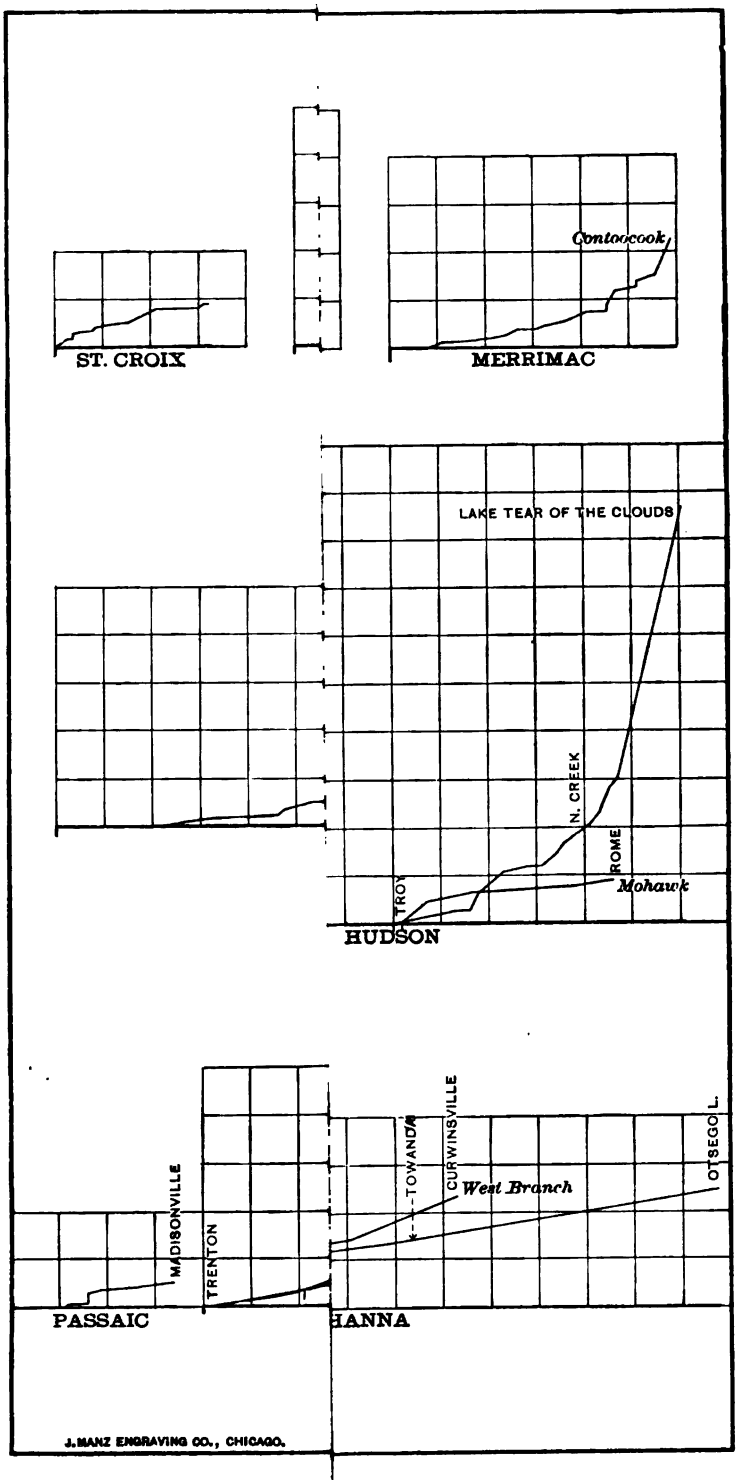

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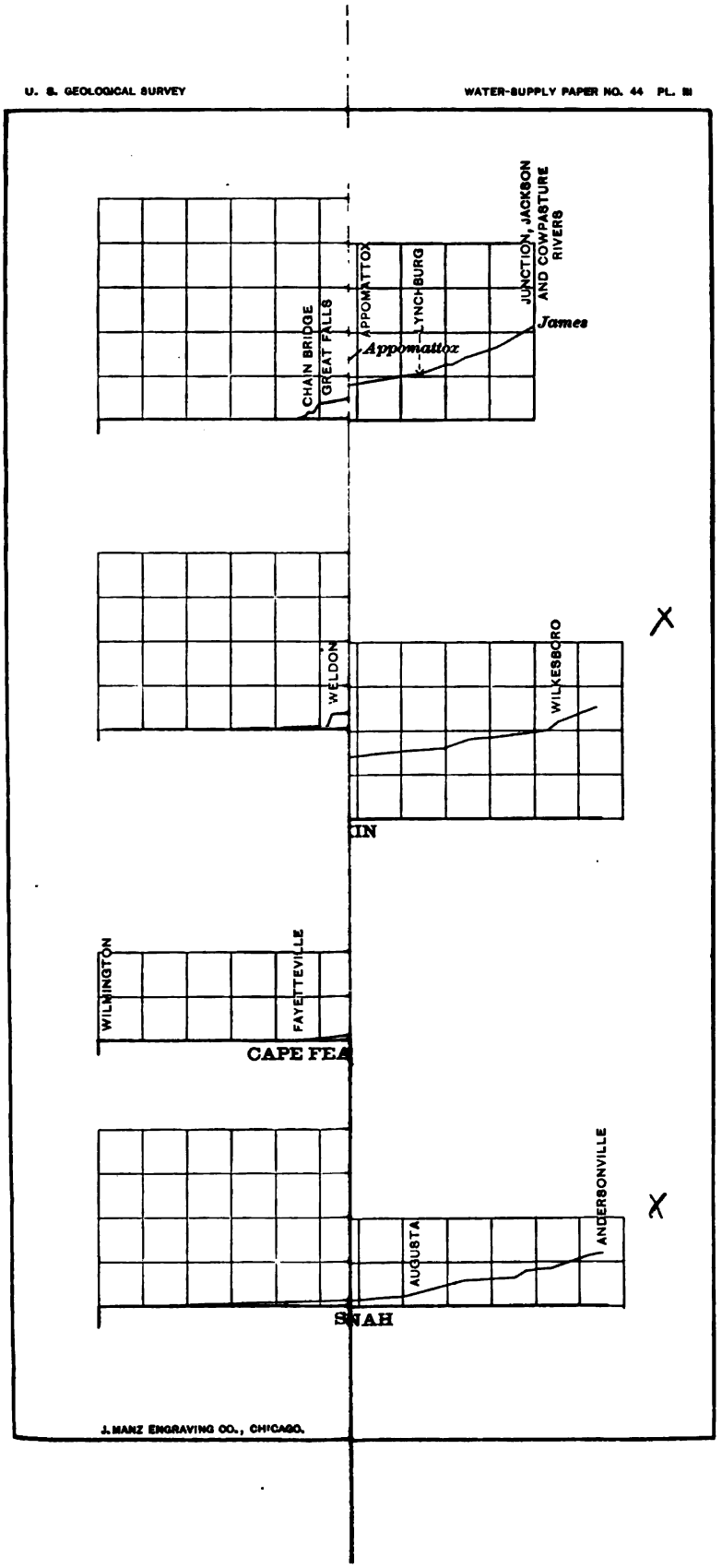
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DEPARTMENT OF THE INTERIOR

WATER-SUPPLY

AND

IRRIGATION PAPERS

OF THE

UNITED STATES GEOLOGICAL SURVEY

No. 45

WATER STORAGE ON CACHE CREEK, CALIFORNIA.—CHANDLER

WASHINGTON
GOVERNMENT PRINTING OFFICE
1901

UNITED STATES GEOLOGICAL SURVEY
CHARLES D. WALCOTT, DIRECTOR

WATER STORAGE

CACHE CREEK, CALIFORNIA

BY

ALBERT E. CHANDLER

WASHINGTON
GOVERNMENT PRINTING OFFICE
1901

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LETTER OF TRANSMITTAL.

DEPARTMENT OF THE INTERIOR,
UNITED STATES GEOLOGICAL SURVEY,
DIVISION OF HYDROGRAPHY,
Washington, D. C., January 24, 1901.

SIR: I have the honor to transmit herewith a manuscript prepared by Prof. Albert E. Chandler, of the University of California, giving the results of a study of the storage possibilities of Cache Creek Basin, California, and to recommend that it be printed in the series of Water-Supply and Irrigation Papers.

The investigations upon which this report is based are a part of a series carried on under the general direction of Mr. J. B. Lippincott, Mem. Am. Soc. C. E., resident hydrographer in California. Mr. J. H. Quinton, Mem. Am. Soc. C. E., has assisted, in the capacity of consulting engineer, in the preparation of the plans and estimates. For this survey financial assistance was rendered by the California Water and Forest Association and by the Woodland Chamber of Commerce. Field work consisted of a reconnaissance throughout the entire drainage basin—all localities being visited which were considered suitable for reservoir sites—and of measurements of the flow of the stream and its various tributaries. The time spent in the field was from June 21, 1900, when a start was made from Woodland, to July 28, 1900.

Eleven years prior to the time of this reconnaissance, or in 1889, a topographic map of Clear Lake and its outlet was made by the Irrigation Survey, the results of that survey being printed in the Thirteenth Annual Report of the United States Geological Survey, Part III, beginning on page 405. At a later period various power companies made studies of the outlet of the lake, and in 1888 the Southern Pacific Railroad made a preliminary survey from Rumsey up Cache Creek Canyon to the lake. A considerable portion of the data accumulated by those surveys has been assembled and employed in this report.

Acknowledgment is due to Mr. Ferdinand Formhalz, of the old Clear Lake Power Company; to Mr. Richard Wylie, of the new power company; and to the engineering office of the Southern Pacific Railroad Company, for assistance given. Thanks should also be extended to the many recorders of rainfall and lake levels who gratuitously

furnished what data they had, and to Mr. J. M. Wilson, of the United States Department of Agriculture, who, during the year 1900, made a thorough study of the water rights and the irrigation systems of Yolo County. Mr. Wilson kindly furnished the data bearing upon the pumping plants, as well as much information concerning the ditches herein described.

Very respectfully,

F. H. NEWELL,
Hydrographer in Charge.

Hon. CHARLES D. WALCOTT,
Director United States Geological Survey.

WATER STORAGE ON CACHE CREEK, CALIFORNIA.

By ALBERT E. CHANDLER.

LOCATION AND PHYSICAL FEATURES.

ihe Creek, the only known outlet of Clear Lake, is situated in County, California, about 75 miles north of San Francisco. the lake the creek flows easterly to the corner of Lake, Colusa, Yolo counties, thence across Yolo County to the sink 8 miles east of Woodland.

ırteen miles from the lake the main stream is joined by the ı Fork, often called Cache Creek. This branch drains the coun- the north and east of the lake, and adds very materially to the age area of the creek, but it is much smaller than the stream the lake and should be considered a tributary.

ı drainage area of the main creek and its branches has been esti- l by the California State engineering office as 1,192 square miles.

TOPOGRAPHY.

ı country in Lake County tributary to Cache Creek is exceed- mountainous and rugged. The peaks to the north of Clear rise to an elevation of 6,000 feet, and their slopes, as well as of the lower ranges, may well be termed precipitous. Pl. I is a ıf the basin. On the northern slopes of the ranges near the lake agnificent belts of fir, white oak, and yellow pine. Elsewhere, ver, the only evidence of vegetation is a dense growth of grease- and chaparral. The interlaced branches of these shrubs make ss through them painfully difficult, but offer little resistance to ing rain water.

m the lake Cache Creek flows through an open country for 5 with a grade of 4.53 feet to the mile. It then follows Cache Canyon for 25 miles. For the first 9 miles of its course through myon it has an average grade of 40.33 feet to the mile, varying 20.41 feet to 137.94 feet. For the remaining distance the aver- ope is about 30 feet to the mile. A view of the bed of the creek

at the head of the canyon is shown in Pl. II, *A*, and a view of the creek in canyon is shown in Pl. II, *B*.

The mountains which form the walls of the canyon rise several hundred feet above the stream, and in many places the immediate banks of the creek stand as perpendicular cliffs 300 feet in height. The bed is generally rendered very rocky by outcropping sandstone and serpentine and the resulting bowlders. At a few places in the canyon settlers have taken advantage of existing benches to raise small crops of corn and hay, but in no place is there a basin which might be used effectively for storage purposes.

After leaving the canyon the creek enters Capay Valley, in Yolo County, through which it winds for a distance of 28 miles, with a grade of 9.5 feet to the mile. Capay Valley is 20 miles long and from 1 mile to 3 miles wide. It is hemmed in on all sides by hills, to the shelter of which is due its remarkable productiveness. Once known merely as a rich wheat-raising district, it is fast becoming famous for its early fruits. All stone fruits seem to thrive there, and in the upper end of the valley even oranges are profitably cultivated on a small scale. From Capay Valley the creek enters the greater Sacramento Valley and flows, with a slope of 4 to 6 feet to the mile, to Tule Basin, bordering Sacramento River. There its natural channel ceases, and its waters enter a canal which connects the sink with the Tule canal. The latter was built by the State of California in 1864 to drain Tule Basin. It empties into Big Lake in the extreme southeast corner of Yolo County, and from the lake its waters reach the Sacramento through a slough.

The soil of Yolo Basin is for the most part a rich alluvial deposit, small areas of adobe being distributed with no uniformity. Mr. J. M. Wilson, of the Department of Agriculture, is of the opinion that "there are 75,000 acres of good fruit and alfalfa land about which there is no question as to the use of water. There is much more that, while it will not bear the reckless use of water, would, with proper drainage and care in application, be greatly benefited."

PRECIPITATION.

Copies of all of the rainfall records kept in Cache Creek Basin were obtained and are given further on. Only one record of the North Fork region has been kept, and that for only the last three years. Short as that record is, however, it gives an excellent idea of extreme variation, since the precipitation for 1899 was almost as unusually large as that for 1898 was unusually small. Unfortunately, the records given are not all continuous. The missing records are as follows: 1888, 1889, and 1890 at Kono Tayee; 1887, 1888, 1890, 1893, 1894, and 1895 at Rumsey; and 1891 at Esparto. The averages would necessarily be changed somewhat were the records complete.

A. HEAD OF CACHE CREEK CANYON.

B. CACHE CREEK IN CANYON.

The table on the next page gives the elevation, the length of record, and the maximum, minimum, and average precipitation for each station. In fig. 1 is plotted the annual rainfall for Kono Tayee, Lakeport, Rumsey, Capay, and Woodland. The difference in the records for Kono Tayee and Lakeport is explained by their locations. Both are situated on Clear Lake, but the former is sheltered by the high bluffs

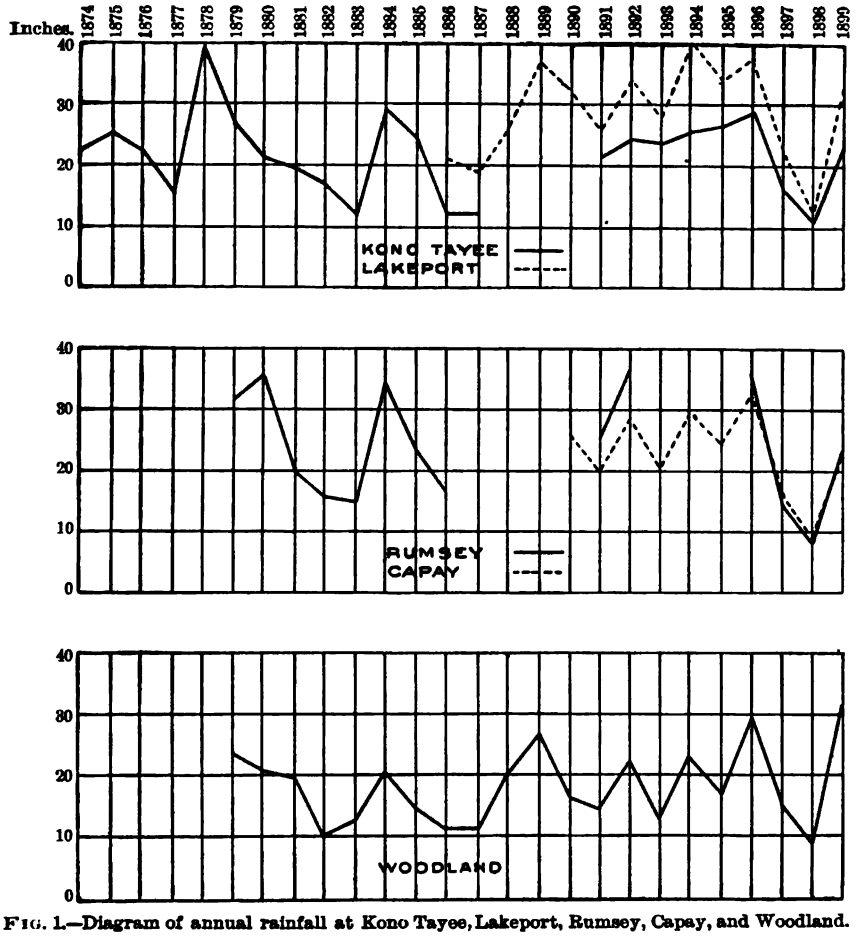


FIG. 1.—Diagram of annual rainfall at Kono Tayee, Lakeport, Rumsey, Capay, and Woodland.

at the narrow central portion of the lake, while the latter is fully exposed at the upper end. Rumsey is at the extreme head and Capay at the extreme foot of Capay Valley, and Woodland is in the lower end of the basin, near Sacramento River. Fig. 2 shows graphically the mean monthly precipitation for Kono Tayee, Rumsey, and Woodland.

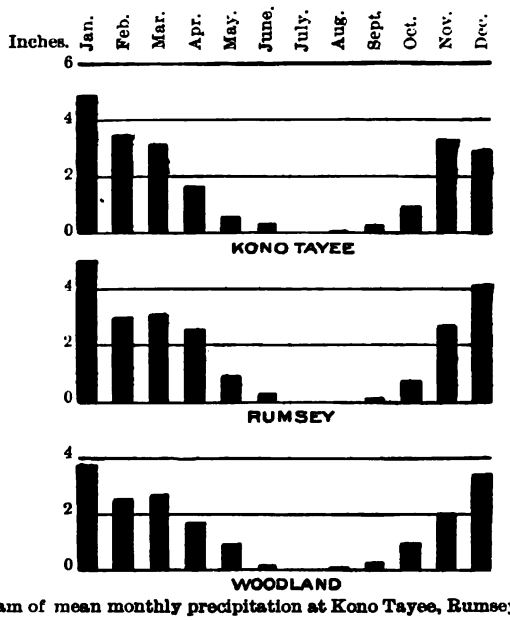


FIG. 2.—Diagram of mean monthly precipitation at Kono Tayee, Rumsey, and Woodland.

Precipitation in Cache Creek drainage basin.

Station.	Elevation.	Length of record.	Maximum rainfall.		Minimum rainfall.		Average rainfall.	Estimated mean annual rainfall.
			Year.	Amount.	Year.	Amount.		
	Feet.	*Years.*		*Inches.*		*Inches.*	*Inches.*	*Inches.*
Bartlett Springs	2,340	3	1899	47.67	1898	17.35	29.71	36.62
Lakeport	1,335	14	1894	40.85	1898	13.50	28.81	29.57
Kono Tayee	1,325	23	1878	39.53	1898	10.89	21.66	.....
Kelseyville	1,425	6	1896	39.88	1898	14.73	27.04	26.85
Rumsey	425	14	1892	36.31	1898	7.96	23.92	24.58
Capay	220	10	1896	32.77	1898	8.68	22.79	21.73
Esparto	192	10	1896	28.45	1898	6.01	18.63	17.39
Woodland	63	22	1899	31.35	1898	9.00	18.25	.....
Yolo	79	6	1894	22.88	1898	5.32	16.99	14.83

The last column of the foregoing table gives the estimated mean annual rainfall at Bartlett Springs, Lakeport, and Kelseyville for a period of twenty-three years, and at Rumsey, Capay, Esparto, and Yolo for a period of twenty-two years.

Monthly precipitation, in inches, at various places in Cache Creek drainage basin.

BARTLETT SPRINGS.

[Observer, Bartlett Springs Hotel.]

Year.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1897	3.50	6.78	5.36	0.75	0.00	0.00	0.00	0.00	0.00	1.70	2.31	3.72	24.12
1898	1.53	8.36	0.19	0.63	2.39	1.10	0.00	0.00	0.00	0.00	1.87	1.28	17.35
1899	13.14	0.00	8.49	0.85	2.47	0.00	0.00	0.00	0.00	4.79	10.06	7.87	47.67
1900	7.51	1.27	6.42	3.23	0.88	0.00	0.00	0.00	.....	.....	.....	.....	.....
Average for 3 years	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	29.71

Monthly precipitation, in inches, at various places in Cache Creek drainage basin—Continued.

LAKEPORT.

[Observers, A. M. Reynolds (1886-1899), Captain Rumsey (1888-1899), and Mr. Force (1898-1899).]

Year.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Annual.
1886	8.18	0.00	1.53	4.80	1.02	0.00	0.00	0.00	0.00	1.14	0.50	3.17	20.34
1887	1.48	8.19	1.89	2.57	0.20	0.11	0.00	0.00	0.14	0.00	1.16	3.03	18.77
1888	6.74	2.16	3.08	0.16	0.57	0.94	0.06	0.00	0.93	0.00	5.56	6.32	26.52
1889	1.36	0.46	8.70	0.88	2.91	0.16	0.00	0.00	0.00	8.16	3.74	11.60	37.97
1890	11.68	4.62	9.25	1.69	0.69	0.00	0.00	0.00	0.45	0.00	0.00	3.68	32.06
1891	1.14	10.91	1.32	2.26	0.80	0.50	0.00	0.00	1.04	0.25	0.28	6.70	25.20
1892	4.63	2.78	2.60	2.52	3.38	0.14	0.00	0.00	0.00	1.14	7.92	8.82	33.93
1893	3.49	5.67	6.78	2.22	0.76	0.00	0.00	0.00	0.68	0.69	4.42	2.80	27.51
1894	10.69	7.37	1.78	1.54	1.39	1.46	0.00	0.00	0.61	1.98	0.92	13.11	40.85
1895	15.66	4.01	4.01	1.16	1.39	0.03	0.01	0.00	1.15	0.01	2.02	4.10	33.55
1896	14.60	0.54	2.75	5.46	2.07	0.00	0.00	0.78	0.31	0.87	4.17	6.48	38.03
1897	3.45	7.92	4.50	0.44	0.07	0.69	0.02	0.00	0.05	1.61	1.63	2.07	22.45
1898	0.58	6.05	0.15	0.58	1.33	0.52	0.00	0.00	0.64	0.94	1.18	1.53	13.50
1899	8.69	0.04	6.53	0.49	0.44	0.38	0.00	0.03	0.00	4.07	6.57	5.47	32.71
Average for 14 years	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	28.81

NOTE.—The figures for the years 1887 to 1892, inclusive, are the mean of two records; those for the years 1893 to 1899, inclusive, are the mean of three records.

KONO TAYEE.

[Observers, Captain Floyd and F. H. Porter.]

Year.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Annual.
1873	.....	.....	.....	.....	.....	.....	.....	.....	0.00	0.00	0.00	0.00	.....
1874	1.88	3.60	4.62	2.05	0.45	0.00	0.00	0.00	0.00	3.70	6.25	0.29	22.84
1875	9.16	0.38	0.92	0.00	0.84	0.42	0.00	0.00	0.00	1.17	6.98	5.12	24.97
1876	6.05	4.22	8.34	0.10	0.00	0.00	0.00	0.00	0.00	0.20	3.50	0.00	22.41
1877	3.17	2.81	1.40	0.50	0.00	0.50	0.00	0.00	0.73	1.65	2.23	1.98	14.97
1878	14.16	11.04	4.60	7.14	0.41	0.00	0.00	0.07	0.00	0.41	1.37	0.33	39.53
1879	3.01	3.41	9.15	0.47	0.64	0.00	0.00	0.05	0.00	0.91	3.57	5.72	26.93
1880	6.24	3.85	4.74	0.48	0.25	0.00	0.00	0.00	0.00	0.00	3.54	1.92	21.02
1881	5.50	6.58	0.64	0.95	0.12	0.25	0.00	0.00	0.00	0.63	2.90	1.77	19.34
1882	1.74	3.20	2.34	1.54	0.40	0.00	0.00	0.00	0.42	1.64	4.42	0.88	16.58
1883	1.40	0.60	3.81	0.95	2.41	0.00	0.00	0.00	0.70	0.99	0.39	0.70	11.94
1884	4.17	1.91	5.35	3.88	0.06	4.08	0.00	0.00	0.77	0.66	0.00	8.26	29.14
1885	1.47	0.51	0.00	1.70	0.00	0.43	0.00	0.00	0.63	0.23	15.37	3.51	23.85
1886	5.61	0.00	1.36	3.30	0.61	0.00	0.00	0.00	0.00	1.05	0.19	0.63	12.75
1887	0.63	6.70	0.95	0.00	0.00	0.00	0.00	0.00	0.38	0.39	0.63	2.93	12.61
1888	5.84	1.20	2.75	0.04	0.00	0.38	0.00	0.00	.....	.....	.....	.....	.....
1891	0.94	10.45	1.12	2.56	0.89	0.02	0.00	0.00	0.15	0.42	0.27	4.34	21.16
1892	2.46	2.13	2.68	1.97	1.37	0.62	0.00	0.00	0.00	0.84	6.53	5.57	24.17
1893	3.84	4.32	5.41	2.06	0.59	0.00	0.00	0.00	0.25	0.42	3.92	2.35	23.16
1894	6.92	4.32	0.00	0.91	0.00	0.00	0.00	0.00	0.63	1.22	0.74	10.81	25.55
1895	14.34	2.32	3.13	0.98	1.14	0.00	0.00	0.00	0.71	0.00	1.69	2.12	26.43
1896	9.42	0.25	2.31	4.62	1.72	0.00	0.00	0.28	0.29	0.73	3.34	5.93	28.89
1897	2.86	4.01	3.75	0.09	0.09	0.48	0.00	0.00	0.00	1.24	1.64	2.02	16.18
1898	0.71	3.99	0.11	0.64	1.67	0.44	0.00	0.00	0.51	0.46	1.15	1.21	10.89
1899	7.74	0.00	5.43	0.70	0.45	0.28	0.00	0.00	0.00	2.76	5.56	0.00	22.92
1900	3.10	2.17	2.89	2.32	0.67	0.00	0.00	0.00	.....	.....	.....	.....	.....
Mean for 23 years	4.93	3.50	3.14	1.63	0.61	0.32	0.00	0.02	0.27	0.94	3.31	2.97	21.66

Monthly precipitation, in inches, at various places in Cache Creek drainage basin—Continued.

KELSEYVILLE.

[Observer, W. A. Maxwell.]

Year.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Annual.
1893	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	4.70	2.55	.....
1894	10.30	6.28	2.57	1.15	1.05	1.20	0.00	0.00	0.52	1.28	0.87	12.88	38.10
1895	2.97	2.25	3.55	0.85	0.50	0.00	0.00	0.00	1.40	0.00	1.12	2.48	15.12
1896	15.37	0.96	1.87	6.01	0.65	0.00	0.00	1.30	0.43	1.08	4.50	7.21	39.38
1897	2.79	5.44	3.25	0.55	0.36	1.07	0.00	0.00	0.25	1.25	1.45	2.40	18.81
1898	0.60	5.61	0.00	1.88	1.64	0.40	0.00	0.00	1.00	0.95	1.15	1.50	14.73
1899	8.87	0.05	7.03	0.00	0.35	0.50	0.00	0.00	0.00	4.94	8.15	6.22	36.11
1900	4.52	1.37	4.99	3.03	0.17	.....	.....	.....	.....	.....	.....	.....	.....
Average for 6 years	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	27.04

RUMSEY.

[Observers, Captain Rumsey (1878-1887) and Southern Pacific Railroad Company (1891-1899).]

Year.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Annual.
1878	.....	.....	.....	.....	.....	.....	.....	.....	0.26	0.30	1.20	0.10	.....
1879	3.34	4.18	10.26	2.05	1.67	0.00	0.00	0.00	0.00	0.28	3.05	7.09	31.92
1880	4.57	1.30	2.73	10.74	1.01	0.00	0.00	0.00	0.00	0.00	0.05	15.51	35.91
1881	9.10	2.21	1.13	1.74	1.26	0.00	0.00	0.00	0.00	0.50	0.83	3.89	20.66
1882	2.64	3.85	3.40	1.22	0.13	0.04	0.00	0.00	0.18	0.84	2.90	0.57	15.77
1883	1.64	0.90	5.24	1.42	3.45	0.00	0.00	0.00	0.42	0.86	0.24	0.66	14.83
1884	5.64	3.19	7.11	4.83	0.32	3.03	0.00	0.00	0.27	0.83	0.00	9.33	34.55
1885	1.48	0.67	0.10	1.38	0.00	0.18	0.00	0.00	0.22	0.55	13.33	5.39	22.30
1886	7.85	0.04	1.16	4.17	0.65	0.00	0.00	0.00	0.00	0.69	0.04	1.56	16.16
1887	1.31	8.57	1.56	2.80	0.17	.....	.....	.....	.....	.....	.....	.....	.....
1891	0.87	13.60	1.16	1.92	1.28	0.00	0.00	0.00	0.85	0.00	0.35	4.98	25.01
1892	3.49	2.84	3.02	1.21	2.78	0.00	0.00	0.00	0.00	1.03	10.57	11.37	36.31
1893	4.80	5.42	5.77	1.76	0.77	.....	.....	.....	.....	.....	.....	.....	.....
1896	17.95	0.00	2.10	5.15	0.45	0.00	0.00	0.00	0.00	0.00	4.83	5.45	35.93
1897	3.52	5.70	1.82	0.00	0.00	0.30	0.00	0.00	0.00	1.40	0.00	1.00	13.74
1898	2.31	3.76	0.00	0.25	0.00	0.00	0.00	0.00	0.60	0.09	0.00	0.95	7.96
1899	7.15	0.00	4.75	0.00	0.56	0.28	0.00	0.00	0.00	2.90	2.57	4.62	22.83
1900	4.35	0.15	1.55	1.25	0.25	.....	.....	.....	.....	.....	.....	.....	.....
Mean for 14 years	5.11	3.02	3.14	2.58	0.97	0.37	0.00	0.00	0.18	0.71	2.77	5.17	23.92

CAPAY.

[Observer, H. J. Taber.]

Year.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Annual.
1889	.....	.....	.....	.....	.....	.....	.....	.....	.....	7.44	3.97	12.22	.....
1890	10.39	4.41	4.43	0.88	1.62	0.00	0.00	0.00	0.40	0.00	0.00	3.50	25.63
1891	0.82	10.62	0.54	2.28	0.58	0.15	0.00	0.00	1.06	0.00	0.36	3.72	20.13
1892	3.02	2.76	2.20	0.42	0.00	0.00	0.00	0.00	0.00	0.97	9.88	9.22	28.57
1893	4.14	4.65	5.61	1.46	0.53	0.00	0.00	0.00	0.18	0.18	1.80	1.81	20.36
1894	7.04	2.86	1.11	0.74	1.70	0.39	0.00	0.00	1.75	1.58	0.44	11 79	29.40
1895	13.58	1.46	2.41	1.56	0.61	0.00	0.00	0.00	1.01	0.00	2.42	1.60	24.65
1896	14.31	0.09	2.71	6.60	0.57	0.00	0.00	0.41	0.50	1.49	3.13	2.96	32.77
1897	4.32	3.58	2.81	0.15	0.35	0.00	0.00	0.00	0.15	1.39	0.80	1.59	15.14
1898	0.67	4.07	0.00	0.38	0.71	0.00	0.00	0.00	0.88	0.50	0.26	1.21	8.68
1899	6.60	0.00	5.01	0.09	0.14	0.28	0.00	0.00	0.00	3.04	2.92	3.88	22.56
1900	3.50	0.30	2.07	1.00	0.64	.....	.....	.....	.....	.....	.....	.....	.....
Average for 10 years	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	22.79

Monthly precipitation, in inches, at various places in Cache Creek drainage basin—Continued.

ESPARTO.

[Observer, Thomas Kennedy.]

Year.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Annual.
1888	.....	.....	.....	.....	.....	.....	.....	.....	0.57	0.00	0.48	5.40	.....
1889	0.34	1.00	4.91	0.25	2.30	0.16	0.00	0.00	0.00	6.07	3.13	9.50	27.66
1890	7.38	3.77	3.47	0.85	1.38	0.00	0.00	0.03	0.32	0.00	0.00	3.02	20.22
1891	0.70	10.06	0.39	2.01	0.52	0.20	.....	.....	.....	.....	.....	.....	.....
1892	0.00	1.70	1.63	0.56	1.68	0.00	0.00	0.00	0.00	0.00	6.26	5.61	17.44
1893	4.12	7.22	0.00	0.00	1.65	0.00	0.00	0.00	0.00	0.00	1.45	1.47	15.91
1894	4.47	2.01	0.67	0.38	1.36	0.68	0.00	0.00	1.10	1.40	0.22	8.26	20.55
1895	8.37	1.14	0.90	1.25	0.30	0.00	0.07	0.00	1.15	0.07	0.82	0.87	14.94
1896	12.69	0.00	1.75	4.41	0.00	0.00	0.00	0.60	0.54	1.38	3.36	3.12	28.45
1897	4.70	2.53	2.82	0.20	0.25	0.31	0.00	0.00	0.12	1.88	0.00	1.98	14.79
1898	0.92	1.50	0.16	0.23	1.14	0.00	0.00	0.00	0.43	0.27	1.36	0.00	6.01
1899	6.22	0.00	4.56	0.34	0.50	0.16	0.00	0.09	0.00	2.30	2.50	3.63	20.30
1900	3.81	0.29	1.59	0.94	0.65	.....	.....	.....	.....	.....	.....	.....	.....
Average for 10 years	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	18.63

WOODLAND.

[Observers, Southern Pacific Railroad Company (1878-1899) and Elston drug store (1894-1899).]

Year.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Annual.
1878	8.17	8.36	3.83	1.47	0.65	0.00	0.00	0.00	0.17	0.22	0.89	0.42	24.18
1879	3.03	3.20	4.74	1.91	1.68	0.24	0.00	0.00	0.00	0.21	2.09	3.69	20.79
1880	1.04	1.60	0.93	6.96	0.35	0.00	0.00	0.00	0.00	0.00	0.06	8.77	19.71
1881	4.94	1.86	1.05	1.53	0.00	0.33	0.00	0.00	0.00	0.37	2.08	2.25	14.41
1882	0.89	1.88	1.89	1.37	0.00	0.00	0.00	0.00	0.59	1.38	2.40	0.51	10.91
1883	1.98	0.45	3.13	1.23	4.12	0.00	0.00	0.00	0.44	1.03	0.20	0.29	12.96
1884	3.47	3.24	4.69	3.83	0.00	0.00	0.00	0.00	0.00	1.00	0.00	4.63	20.86
1885	1.43	0.12	0.10	1.44	0.00	0.00	0.00	0.00	0.00	0.20	8.70	3.75	15.74
1886	4.78	0.00	1.31	4.10	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.29	11.48
1887	0.80	5.58	0.65	1.53	0.00	0.00	0.00	0.00	0.00	0.00	0.40	3.30	11.26
1888	4.20	1.27	2.38	0.10	1.10	0.00	0.00	0.00	0.56	0.00	5.57	4.97	20.15
1889	0.00	0.55	6.21	0.62	1.46	0.85	0.00	0.00	0.00	5.32	3.75	8.48	26.74
1890	5.10	2.40	3.35	1.00	1.60	0.00	0.00	0.00	0.60	0.00	0.00	2.35	16.40
1891	0.82	8.08	0.35	1.17	0.43	0.00	0.00	0.00	0.00	0.00	0.40	3.10	14.35
1892	2.05	2.73	2.14	1.28	2.22	0.00	0.00	0.00	0.00	0.57	5.47	6.10	22.56
1893	2.88	2.78	2.00	0.62	0.61	0.00	0.00	0.00	0.00	0.08	1.71	1.92	12.60
1894	4.12	2.15	0.88	0.42	1.49	0.77	0.00	0.00	0.85	1.12	0.85	10.82	23.47
1895	9.83	1.28	0.91	0.52	0.45	0.00	0.00	0.00	1.37	0.19	1.69	0.90	17.14
1896	11.87	0.11	2.08	6.73	0.76	0.00	0.00	0.35	0.50	1.27	3.76	2.03	29.46
1897	3.32	5.21	2.75	0.25	0.29	0.09	0.00	0.00	0.02	1.81	0.56	0.72	15.02
1898	0.43	3.35	0.12	0.21	1.38	0.18	0.00	0.00	0.35	0.98	0.47	1.53	9.00
1899	5.67	0.14	14.46	0.13	0.08	0.78	0.00	0.00	0.00	3.55	3.01	3.53	31.35
Mean for 22 years.	3.67	2.56	2.72	1.75	0.85	0.12	0.00	0.02	0.25	0.92	2.01	3.42	18.25

NOTE.—The figures for the years 1894 to 1899, inclusive, are the mean of two records.

Monthly precipitation, in inches, at various places in Cache Creek drainage basin—Continued.

YOLO.

[Observer, Yolo Orchard.]

Year.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Annual.
1893	.....	.....	.....	0.55	0.78	0.00	0.00	0.00	0.06	0.10	1.56	2.06	.....
1894	4.28	2.23	0.98	0.42	1.51	0.75	0.00	0.00	0.85	1.25	0.80	9.81	22.8[illegible]
1895	9.10	1.48	1.15	0.61	0.41	0.00	0.00	0.00	1.59	0.30	2.07	1.03	17.74
1896	8.49	0.30	1.79	5.81	0.59	0.00	0.00	0.15	0.56	1.66	1.10	1.76	22.21
1897	2.56	4.04	2.11	0.31	0.26	0.20	0.00	0.00	0.07	1.35	0.39	1.25	12.54
1898	0.40	2.72	0.00	0.24	0.26	0.00	0.00	0.00	0.25	0.75	0.42	0.28	5.32
1899	5.81	0.00	3.97	0.36	0.32	0.80	0.00	0.09	0.00	3.23	3.59	3.09	21.26
1900	3.14	0.32	1.38	0.86	0.89	.....	.....	.....	.....	.....	.....	.....	.....
Average for 6 years	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	16.99

STREAM MEASUREMENTS.

Measurements were made of the discharge of all of the small streams in which water was flowing at the time they were visited. The flow of the main stream at its various junctions with other streams, was also recorded, to show the comparative value of the branches. During the latter part of July, 1900, a series of measurements was carried down Cache Creek from Rumsey. A similar series was made by Mr. Wilson a month earlier. The results of both series are given in the table on the next page. Mr. Wilson's measurements show a slight increase in discharge between Rumsey and Capay, but the records taken a month later show a gradual decrease.

Below Capay the creek widens until in some places its gravel bed is 1,000 feet from bank to bank. Above Woodland it narrows again to 100 or 200 feet, and flows between clay banks 30 to 50 feet high. Below Capay part of the flow sinks into the gravel, but it reappears again just above Moore's dam.

The measurements taken in July show discharges as follows: At Capay, 88.1 second-feet; opposite Madison, 54.6 second-feet; above Moore's dam, 69 second-feet. The last measurement taken was on July 24, at Stevens's bridge, 2 miles below Moore's dam. A half mile below the bridge the stream ceased to flow, leaving the gravel bed dry. A few miles farther down the creek water was found standing in pools, the underlying clay stratum having forced it to the surface; but whatever flow existed was within the gravel.

In the following table are recorded the results of the measurements of the main creek and its tributaries:

Discharge measurements of Cache Creek and its tributaries.

Date.	Stream.	Point of measurement.	Hydrographer.	Area of section.	Mean velocity.	Discharge.
1900.				*Sq. ft.*	*Ft. per sec.*	*Sec.-ft.*
June 25	Cache Creek	At Rumsey	A. E. Chandler	78.3	2.413	189.0
June 27	do	At Bear Creek	do	116.8	1.336	156.0
June 29	do	At North Fork	do	68.4	2.360	161.4
July 17	do	At Clear Lake	do	68.0	1.572	106.9
July 20	do	At Rumsey	do	54.3	1.705	92.6
Do	do	At Tancred	do	43.0	2.079	89.4
July 21	do	At Capay	do	57.9	1.522	88.1
July 23	Adams ditch		do			6.8
Do	Cache Creek	At Madison	do	35.9	1.521	54.6
Do	do	At Moore's dam	do	44.0	1.568	69.0
July 24	Moore ditch		do	73.1	0.676	49.4
Do	Cache Creek	At Stevens's bridge	do	22.7	0.899	20.4
June 29	do	At Rumsey	J. M. Wilson			166.8
Do	do	At Tancred	do			167.5
Do	do	Five miles above Capay	do			173.6
June 28	do	At Capay	do			161.6
June 30	do	At Esparto	do			152.7
Do	do	At Madison bridge	do			140.9
Do	Moore ditch		do			60.5
July 3	Cache Creek	At Stevens's bridge	do			75.8
Do	do	At Nelson's bridge	do			53.0
Do	do	At Cache Creek sink	do			51.8
Do	Tule canal	Opposite Woodland	do			29.7
Aug. 20	Cache Creek	At Clear Lake	do			39.6
Aug. 27	do	One-fourth mile above Rumsey.	do			27.6
June 29	North Fork	At mouth	A. E. Chandler	4.7	1.085	5.1
June 30	do	Above Long Valley Creek.	do	4.8	1.333	6.4
June 27	Bear Creek	At mouth	do	1.8	1.000	1.8
June 30	Long Valley Creek	do	do	3.8	0.763	2.9
Do	Wolf Creek	One mile above mouth	do	0.5	1.000	0.5
July 2	North Fork	Above Bartlett Creek	do	6.6	0.500	3.3
Do	Bartlett Creek	At mouth	do	1.9	0.737	1.4
Do	Stanton Creek	do	do	1.3	1.154	1.5
July 8	North Fork	At Little Indian Valley	do	4.4	0.636	2.8
July 12	Scotts Creek	Eight miles above Clear Lake.	do	0.8	0.625	0.5
Do	Middle and Clover creeks.	Near Upper Lake	do	2.1	0.714	1.5
July 16	Kelsey Creek	Two miles above Kelseyville.	do	5.4	0.852	4.6

IRRIGATION WORKS.

That water can successfully be diverted from Cache Creek is proved by the number of irrigating ditches that have been constructed in Yolo County. Only one, however, the Moore ditch, is of present importance. Two others, the Capay Valley ditch and the Adams

ditch, are being used this summer (1900) on a small scale; that they are not in general use is due to protracted litigation over water rights, the common bane of irrigation throughout California.

CAPAY VALLEY DITCH.

The first ditch, in point of position, to take water from Cache Creek is the Capay Valley. The diversion works (see Pl. III, *A*) are situated on the right bank of an old channel of the stream at the very head of Capay Valley. The ditch was begun in 1871, and about 12 miles were constructed before work was stopped permanently in the fall of 1873. The original plan was to irrigate all of Capay Valley on the south side of the creek (about 13,000 acres), but it is probable that not more than 8 miles of the canal were ever used.

It was started with a bottom width of 24 feet, but after the first half mile was narrowed to a width of 16 feet, and at the end of the third mile again contracted to 8 feet. The lower end of the ditch was widened to 10 or 12 feet on the bottom. Indeed the ditch seems to have been characterized by a total lack of system in its construction. The flumes, of which there are six, crossing small tributaries of Cache Creek, are quite as devoid of regularity as the dimensions of earthen channel. They are usually 4 feet deep and vary in width from 8 to 16 feet. They are constructed, strange to say, of Oregon fir throughout, a timber of great strength, but poorly adapted to structures which are alternately wet and dry. As a natural consequence, the flumes have decayed rapidly and are now wretchedly out of repair.[1]

At present (1900) the ditch beyond the first 3 miles is simply an undulation of the surface, and not a trace of the flumes described by Mr. Schuyler is to be found. Very little has been done with the ditch since 1886, when, after three trials in the lower courts, the supreme court affirmed a judgment "*forever* enjoining the company from taking water from Cache Creek at any and all times when such diversion would interfere with the flow of a sufficient quantity (432 cubic feet) in the Moore ditch." This summer (1900) the first 2 miles of the ditch were used by the fruit growers owning farms along it. They took turns in using the water, and in July, when the creek was low, by their combined efforts a diverting weir of brush, straw, and stones was thrown across the creek. No payment whatever is made by them to the company.

TWO PROPOSED CANALS.

In the report just cited Mr. Schuyler describes a dam many of the timber bents of which still stand intact. He says:

In addition to the Capay ditch, the Clear Lake Water Works began another work, which, as projected, was the most comprehensive scheme for the disposal of the waters of Cache Creek ever attempted. It contemplated the construction of two large canals taking their head at a point some three miles above the village

[1] Report on the works and practice of irrigation in Yolo County, by J. D. Schuyler: Report of California State Engineer, 1880, Appendix E, p. 185.

A. DIVERTING WEIR OF CAPAY VALLEY DITCH.

B. DAM AND HEAD OF ADAMS DITCH.

of Capay; the one to irrigate the plains on the north of Cache Creek, the other to cover the lands south of that stream. The latter was to have been navigable and to extend to deep water in Suisun Bay or elsewhere. At the proposed head of the canal a dam was constructed six hundred feet in length. It was made prismoidal in form, 15 feet wide on top, with the upper slope of ¼ on 1, and the lower 1 on 1. The south half of the dam was about 8 feet high and the north half about 13 feet. It was constructed of heavy timbers bolted to the sandstone rock which there cropped out in the channel, and the interior was filled with rock and gravel. The dam is reported to have cost $50,000. The canals were never begun. In all of these various improvements the Clear Lake Water Works have expended some $150,000, but thus far there have been few satisfactory results from their investment.

COTTONWOOD DITCH.

Another ditch, not at present in service, is the old Cottonwood ditch, which heads on the south bank of Cache Creek, a half mile below the dam just described. It was commenced in 1864, and was ultimately extended a distance of 10 miles. The bottom width was 12 feet, the depth 5 to 5.5 feet, and the grade 3 feet to the mile. The original headworks were substantially built and advantageously located, but not a trace of them now remains. The ditch is said to have cost $50,000.

In 1879 the Capay Ditch Company succeeded the Cottonwood Ditch Company. In 1882 the Moore Ditch Company enjoined the Capay Ditch Company from using water from Cache Creek, and the lawsuit which resulted remained in the courts many years, but was never settled. In consequence, the legal status of the ditch was rendered so uncertain that it rapidly fell into disuse.

ADAMS DITCH.

The Adams ditch diverts water from the north bank of Cache Creek less than a mile above Capay. Three miles of it were constructed in 1870, to irrigate 20 acres of Chinese gardens. In the following year the ditch was enlarged to a bottom width of 6 feet, a top width of 13 feet, and a depth of 2½ feet. In 1877–78 a new section was constructed and part of the old channel was abandoned. In 1882 the ditch was extended, under contract with the farmers above Cacheville (Yolo), to supply water to the old Cacheville agricultural ditch. The latter ditch was constructed in the winter of 1859–60, but has been abandoned since the destruction of its headworks by a flood late in the sixties. Just after the construction of the extension alluded to the Adams ditch became involved in the courts with the Moore ditch and the extension was never used. This summer (1900) the first 3 miles of the ditch were utilized to irrigate 60 acres of alfalfa and 20 acres of vegetables on the Adams place. The ditch has no headworks, the water being diverted into the open channel by a temporary dam of brush, straw, earth, and stones. (See Pl. III, *B*.) On July 23, 1900, there were 6.8 second-feet of water in the ditch.

MOORE DITCH.

The Moore ditch, the oldest and most important ditch on Cache Creek, was built in 1856. It heads 8 miles above Woodland, and originally was 8 feet wide on top and 2 feet deep. In 1863 it was enlarged to a bottom width of 16 feet. In 1881 the grade was changed from 1.34 to 2.34 feet to the mile, and by extending the ditch upstream a gain of 10 inches in the fall was secured. At the same time a timber dam was constructed across Cache Creek, 24-foot piles being driven into the stream bed. Prior to the construction of this timber dam temporary dams of brush and earth were constructed as soon as the water became low in summer, but they were carried out by the first fall flood. The timber dam was swept away by the storm waters of 1889–90, and has not been replaced.

At present the main ditch and its branches have a total length of about 80 miles. The branches are under the management of subcorporations, the two most important being the South Fork Water Ditch Company and the Farmers' Irrigating Ditch Company. The ditch of the former was begun in 1864, and now has a length of 4 miles and a capacity of 40 second-feet. It is V-shaped, 20 feet wide on top, 4 feet deep, and has a grade of one-half foot to the mile. It has a capacity to irrigate 2,000 acres; it has irrigated 1,000 acres. The original cost was $2,400; but it could be constructed now for $800. The cost of irrigating varies from 50 cents to $2 an acre. The branch under the Farmers' Irrigating Ditch Company was also started in 1864. It is 1¼ miles long and controls 4 miles of laterals. The top width of the ditch is 16 feet, the bottom width 8 feet, and the depth from 2½ to 4 feet. Its capacity is 20 second-feet, and 1,000 acres are irrigated by it. The original cost was $2,400.

The maximum rate at which water may be sold is fixed by the board of supervisors of Yolo County at "$4 per foot for the period of 24 hours, with the water flowing in at the rate of 2 feet per second. * * * The measurements shall be made from the top of a weir 4 inches high, constructed in the bottom of the bulkheads where measurement is made, and over which weirs such water must flow."

Although a capacity of 432 second-feet is claimed for the Moore ditch, it seems impossible that it ever could have carried half that amount. The banks have been allowed to silt up, and now are covered by a luxuriant growth of water weeds, which necessarily reduces the efficiency of the canal to a minimum. The headworks are substantial and are admirably situated. (See Pl. IV, *A*.) It is said that even when the creek is dry through Capay Valley there is a supply at the headworks. It is deplorable that a durable diverting dam has not been built, for the temporary dam of earth and brush is not constructed until June, and as a result the farmers are not able to reap the benefits of late spring flooding.

A. HEADWORKS AND DAM OF MOORE DITCH.

B. HEADWORKS OF LANGENOOR DITCH.

LANGENOOR AND HENNIGEN DITCHES.

Below the Moore ditch there are two private ditches—the Langenoor, which heads on the north bank of the creek a half mile below Nelson's bridge, and the Hennigen, located on the south side a mile above the canal connecting Cache Creek with the Tule canal.

Regarding the construction of the Langenoor ditch Mr. J. D. Schuyler writes:

> The ditch was constructed and first used in 1864 to irrigate 20 acres of grain. * * * At its head there is an underlying stratum of clay in the creek bed, at a depth of but 4 feet, which brings water to the surface. To force all of the water to top a double row of sheet piling, about 40 feet apart and 100 feet long, has been driven into bed of creek, covered with a floor even with the surface. In connection with this arrangement was a low, movable dam, intended to be raised during low water and removed when the floods came.

A few of the upright posts of the dam and a part of the wooden floor now remain. At the head of the ditch is an old wooden head gate, or bulkhead, as it is called, which was constructed in 1877. (See Pl. IV, *B*.) It is 20 feet wide and 12½ feet high, and has two sets of gates (four gates in each set) 18 feet apart. The gates are made of 3-inch timber, and are 4 feet wide by 3½ feet high. In 1888 a wall of masonry 16 feet high was constructed across the ditch 140 feet from the old head gate, and the ditch between the two was lined with a wooden flume. Through the base of the masonry there are four channels, each 3.8 feet wide by 2.6 feet high, which are closed by means of wooden gates. The ditch is a half mile long, 24 feet wide on top, 10 feet wide on bottom, and 5 feet deep. In April, 1900, it was used to irrigate 100 acres of alfalfa and 70 acres of fruit. As the creek at this point is dry by the end of July, the ditch can be used only in spring and early summer.

The Hennigen ditch was constructed in 1887. It is 1 mile long, 30 feet wide on top, 10 feet wide on bottom, and 2½ feet deep. It has a capacity of 15 second-feet, and controls 700 acres, 200 of which it has irrigated. The original cost was $2,200; up to the present time $4,400 has been expended on it.

PUMPING PLANTS.

Not being able to rely upon the irrigating ditches, many of the farmers about Woodland have resorted to pumping from Cache Creek and from wells. Now that their pumping plants are established, most of the operators find them so effective that they would hesitate to abandon them for even an improved system of ditches.

Outside of the vicinity of Woodland the only pump used for irrigation this season was one in charge of E. F. Hoswell, of Rumsey, in Capay Valley. The pump and engine are mounted on a wagon, and are moved from farm to farm as necessity requires. The engine is an 18-horsepower gasoline engine, known as the Hercules. Its cost

was $1,000, and it uses 22 gallons of distillate, costing 10½ cents a gallon, in a working day of ten hours. The pump is of the centrifugal pattern, and has a capacity of 700 gallons a minute. Its cost was $150. Water is pumped directly from the creek and forced through 8-inch pipe to the field to be irrigated, and often it is elevated to a height of 55 feet. The average cost of operation is $1 an acre. A view of this portable pumping plant is shown in Pl. V, *A*, and a view of the pumping plant of Robert Morrison in operation on Cache Creek is shown in Pl. V, *B*.

In the Woodland district there are not fewer than twenty places where pumps are now used. Eight of these depend upon Cache Creek for their supply. As the pumping plants are all below Moore's dam, the supply is generally insufficient after the middle of July. When visited in the later part of July, 1900, the pumps on the creek could be used only a half day at a time. There was no water flowing in the creek, but long pools had formed where the water, percolating through the gravel, was forced to the surface. It was from depressions in these pools that the pumping was done, and when the supply was exhausted the pumps were stopped until the pool filled again. In some places earthen dams 1 foot or 2 feet high were thrown across the bed to hold the water, and small ditches were dug between the pools above to let more water down.

Those who use wells report an unlimited supply. In April, 1897, Mr. C. S. Mering pumped eighteen days and nights with a pump having a capacity of 3,000 gallons a minute, and noticed no diminution in the supply. Mr. R. B. Blower has used his well since 1879, and it has never failed to furnish a supply.

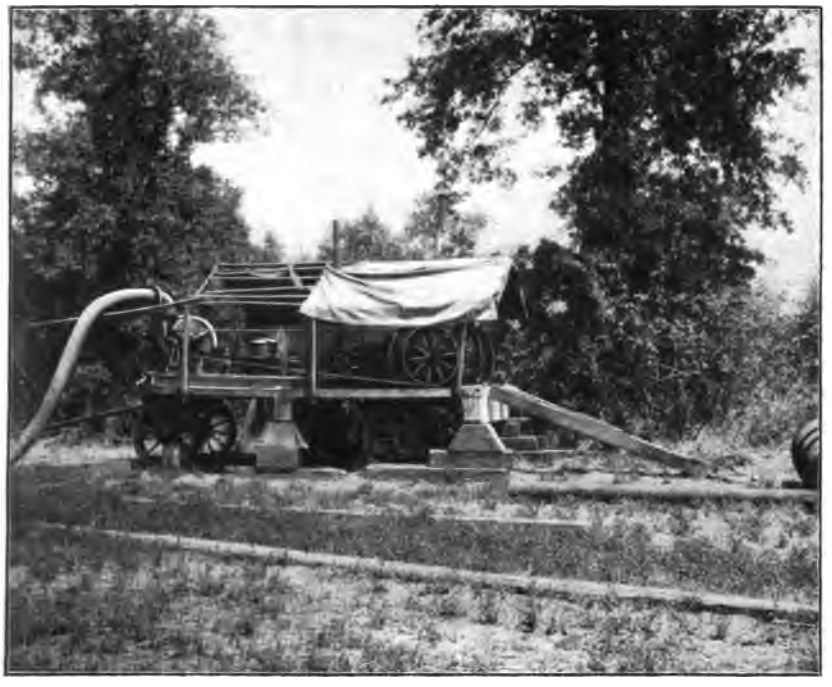
With the exception of the gasoline engine of Mr. S. V. Scarlet, all of the engines in use are steam engines. Straw, brush, wood, and coal are used for fuel. The average price paid is 75 cents a load for straw, $4 a cord for wood, $8.50 a ton for coal, and 11 cents a gallon for gasoline.

Mr. S. V. Scarlet's engine is portable, and is used by nine of his neighbors. In some cases the pumps are used to furnish water to fields adjacent to lands of the owner as well as to his own fields. The table in the following discussion on underground waters gives the more important details regarding these pumping plants.

UNDERGROUND WATERS.

DEPTH OF WATER PLANE.

As may be seen in the following table, the depth at which water may be found at the pump sites varies from 10 to 25 feet. It is not unusual, however, for the depth at two points on a comparatively level farm to differ considerably more than this.



A. PORTABLE PUMPING PLANT IN CAPAY VALLEY.

B. PUMPING PLANT ON CACHE CREEK.

Irrigation pumping plants in Woodland district.

Owner.	Source of supply.	Acres irrigated.	Size of well.		Depth to water.	Height to which water is elevated.	Engine.		Pump.			Operating expenses per acre.c
			Diameter.	Depth.			Horse-power.	Cost.	Diameter.	Capacity.	Cost.	
			Ins.	*Feet.*	*Feet.*	*Feet.*			*Ins.*	*Gallons per minute.*		
Byron Jackson	6 wells	160	8	78	18	18 to 44	60	$1,600	....	3,000	$500	$1.50
R. B. Blower	Well	80	18	25	10	10 to 24	25	230	6	2,600	.....	.83
J. R. Fisher	3 wells	10	2-12, 1-8	60	24	28	17	a 600	6	1,800	213	3.20
J. E. Scarlet	....do	80	12	42	16	28	25	.....	8	2,000	214	1.50
S. V. Scarlet	2 wells	20	12	80	21	31	25	1,675	6	3,000	150	1.00
C. H. Steinberg	....do	80	24	45	15	25 to 30	25	a 700	6	.....	160	1.57
C. S. Mering	3 wells	18	10	45	16	30	35	.....	8	3,000	450	.75
F. Martinelli	....do	30	5	.....	20	30	16	500	....	.....	.....	2.00
L. Cramer	Well and Cache Creek.	25	24	55	25	25 to 35	20	1,800	10	7,000	550	.75
O. J. Adams	Cache Creek.	13	.....	.....	.....	27	8	250	....	400	125	.83
Mrs. Peart	....do	70	.....	.....	.....	20	(b)	(b)	12	5,000	.....	1.20
W. W. Nelson	....do	80	.....	.....	.....	11	45	2,200	10	5,000	250	2.57
F. Sanderson	....do	17	.....	.....	.....	20	(b)	(b)	5	1,200	250	2.00
Robert Morrison	....do	10	.....	.....	.....	25	(b)	(b)	6	2,000	100	1.50

a Secondhand. b Use S. V. Scarlet's gasoline engine. c One irrigation.

This spring (1900) the firm of Botsford & Blithroad bored a number of wells along the roads leading from Woodland, for the use of watering carts. The depths to first water and gravel in thirteen of these are as follows:

Well borings near Woodland.

Boring.	Depth to water.	Depth to gravel.
	Feet.	*Feet.*
No. 1	16	32
No. 2	18	38
No. 3	20	20
No. 4	24	114
No. 5	30	40
No. 6	30	3
No. 7	19	.....
No. 8	19	.....
No. 9	19	46
No. 10	10	36
No. 11	7	24
No. 12	3.5	20
No. 13	7.5	.....

In borings Nos. 7 and 8 no gravel was encountered, although the borings extended to a depth of 92 feet. The average depth to water of these 13 borings is 17 feet—a depth which the experience of other well borers has shown to be a good average for the irrigable lands about Woodland. In 1883–84 borings were made by engineers to determine the depth of the water plane through Hungry Hollow, a depression just northwest of Moore's dam, the results being submitted in the case of Moore *v.* Adams. The following depths have been taken from the profile of the 1884 borings, the lines starting at Cache Creek above Moore's dam and running 5 miles into Hungry Hollow:

Borings to determine depth of water plane through Hungry Hollow.

Distance from creek.	Depth of water plane.
Miles.	*Feet.*
1	14
2	14
3	16
4	19
5	23.5

NOTE.—The water plane was reported to be 1.2 feet higher in 1883 than in 1884.

ARTESIAN FLOW.

There is only one artesian well in Yolo County. It is on the Dinsdale place, 1 mile east of Woodland. It has a depth of 134 feet and a diameter of 5 inches. When visited during the latter part of July, 1900, the discharge was very small, but it is said to be considerable during the wetter seasons.

TRIBUTARIES IN CAPAY VALLEY.

The hills on the right and left of Capay Valley are cut up into a series of small canyons, with sharp ridges between. Consequently, the watersheds are so small and the slopes so steep that little or no water is to be found in the canyons after the rains have ceased. The most important of these are Rumsey Canyon, at the extreme head of the valley, and Cadenasso Canyon, near its foot, both on the west side. On June 23, 1900, there was a trace of a stream in the former canyon to a point 2 miles above its mouth, where the water sank, leaving the rest of the bed dry. During the summer water stands in small pools in Cadenasso Creek after it reaches the valley, the bed above being dry. The slope of the beds of both canyons is so great and the width so small that even if a sufficient drainage area existed they could not profitably be utilized for storage purposes.

FISKE AND DAVIS CREEKS.

These creeks are usually marked on the maps of this region. Both enter the main creek, from the south side, in Cache Creek Canyon. Like the smaller streams in Capay Valley, they are dry soon after the last rains, and their beds remain dry washes until the next wet season.

BEAR CREEK.

Other than the North Fork, the only tributary to Cache Creek of any consequence is Bear Creek, which enters the main stream on the north side, 5 miles above the mouth of the canyon. Twelve miles up the creek is Bear Valley, which has a length of 10 miles and an average width of 2 miles, and in which good crops of hay and grain are raised. Near the lower end of the valley the surface is undulating, and nowhere is there a good dam site, even if the great amount of water required to flood this large area were obtainable.

Below the valley there is very little open country, and for the last 5 miles the creek traverses a narrow canyon. Two miles below Bear Valley, Sulphur Creek empties into Bear Creek. Sulphur Creek drains a district in which three large quicksilver mines are operated, and many sulphur and mineral springs exist in its own bed. By the time it reaches Bear Creek it has dissolved so much mineral matter that it renders the water of the larger creek brackish and salty, unfitting it for irrigation purposes.

When measured at its mouth on June 27, 1900, Bear Creek had a discharge of 1.8 second-feet, while on the same day Cache Creek had a discharge of 156.0 second-feet. Bear Creek is said rarely to become dry. Even in the fall of 1898, when there was no outflow from the lake, water was running in this branch.

NORTH FORK.

The North Fork of Cache Creek rises in the mountains north of Bartlett Springs. The region of its headwaters is famous for its many springs, the medicinal properties of which attract hundreds of visitors every summer. Bartlett Springs is situated on Bartlett Creek, which flows into the North Fork 3 miles below the springs. On July 2, 1900, there was a discharge of 1.4 second-feet in Bartlett Creek and a discharge of 3.3 second-feet in the North Fork at their junction. Seven miles below the junction the North Fork leaves the mountains and enters Little Indian Valley. As early as July 2, 1900, the stream in the upper part of the valley was dry, the water having sunk into the gravel floor. Two miles from the head of the valley rock outcrops in the stream bed, forcing the water to the surface. At that point Stanton Creek joins the North Fork. On July 2, 1900,

Stanton Creek had a discharge of 1.5 second-feet, and on July 8 the discharge of the North Fork at the foot of the valley measured 2.5 second-feet.

At the foot of Little Indian Valley the North Fork enters a narrow canyon, through which it flows for a distance of 5 miles. A half mile below the lower end of the canyon Wolf Creek joins it. On June 30, 1900, the latter stream was dry, except for a few standing pools, for a distance of a mile above its mouth, at which point it had a discharge of 0.5 second-foot.

A mile below Wolf Creek and 7½ miles above Cache Creek is the mouth of Long Valley Creek. On June 30, 1900, this stream had a discharge of 2.9 second-feet. On the same day the discharge of the North Fork at that point was 6.4 second-feet.

Below Long Valley Creek there are no streams tributary to the North Fork. There are numerous canyons down which the storm waters rush after heavy rains, but none in which water can be found in the summer. The North Fork itself generally has no running water in its lower reaches after July. The water stands in pools, between which it seeps through the gravel. These pools remain throughout the dry season, and afford the only drinking water for the cattle in the neighborhood. For the last few miles the gravel floor is often 500 feet wide, and from it the greatly eroded clay banks rise to a height of from 50 to 100 feet. A view of these eroded clay slopes is shown in Pl. VI, *A*, and a view of the gravel bed of the creek in Pl. VI, *B*.

On June 29, 1900, the discharge at the mouth of the North Fork was only 5.1 second-feet, while on the same date the discharge of Cache Creek at the junction was 161.4 second-feet.

RESERVOIR SITES ON NORTH FORK.

The only basins in the North Fork system which might be mentioned as reservoir sites are Twin Valleys, Little Indian Valley, and Long Valley.

The Twin Valleys are situated on a short branch of the North Fork a few miles north of Bartlett Springs. They are separated by a canyon a half mile in length. Each valley is three-fourths of a mile long and a half mile wide. As the narrow canyon could easily be dammed, the upper valley is an ideal site for a storage basin; but as the entire catchment area above the valleys is only 5 square miles, there would be very little water for storage purposes, and they can not, therefore, be recommended as reservoir sites. Both valleys are under cultivation, hay and garden produce for the hotels being raised, the vegetables being irrigated by water from the creek.

Long Valley is situated on Long Valley Creek, 2 miles from its mouth. It is 6 miles long, and has an average width of 1,000 feet

A. NORTH FORK OF CACHE CREEK, SHOWING ERODED CLAY SLOPES.

B. GRAVEL BED OF NORTH FORK OF CACHE CREEK.

and a grade of 25 feet to the mile. Although Rice's map of Lake County shows a drainage area of about 30 square miles, the run-off during the drier years, when water is most needed, is stated, on excellent authority, to be very small, as might be expected from experience elsewhere in the same region. Moreover, the valley at its foot, the so-called Narrows, is 700 feet wide. Even should an ample supply be assured, the cost of a dam across the Narrows would be almost prohibitive. There is some hay and grain raised in the valley, but the income of its residents is largely derived from stock raising.

Little Indian Valley, the only basin on the North Fork to be recommended as a reservoir site, is 5 miles long and averages from a half to three-fourths of a mile in width. Almost half of the entire area is under cultivation, but the soil contains so much gravel that the grain crops are very light. A small portion in the middle of the valley and near the creek is irrigated for vegetables, a temporary earthen dam being thrown across the stream at a point where the rock outcrops. A view of the valley looking toward the dam site is shown in Pl. VII, *A*, and views of the dam site are shown in Pls. VII, *B*, and VIII, *A*.

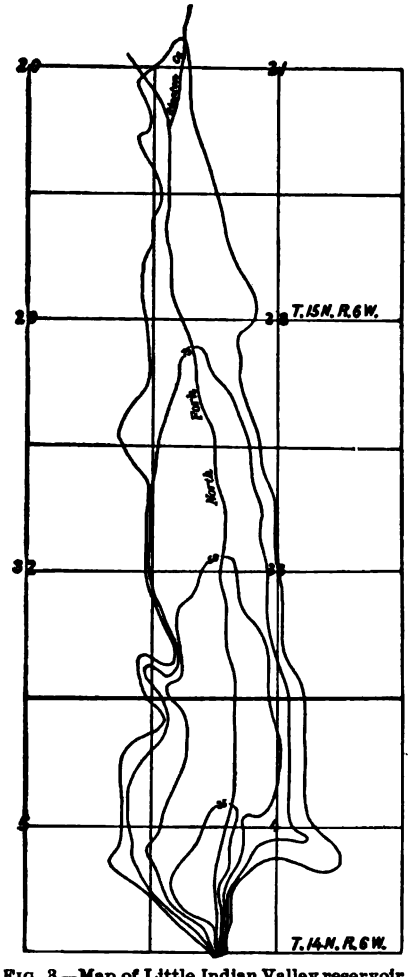


FIG. 3.—Map of Little Indian Valley reservoir site.

A reconnaissance survey of the valley was made to determine its approximate storage capacity, contours being located at intervals of 25 feet, up to the 100-foot level. The following table shows the acreage of each of the four planes, and the storage capacity, in acre-feet, at the intermediate layers, also the total capacity for each contour. The area of the 125-foot contour is estimated to be 1,500 acres, which is considered a minimum figure, as the valley widens very appreciably at its upper end.

Storage capacity of Little Indian Valley reservoir site.

Contour.	Area.	Capacity.	Total capacity.
	Acres.	*Acre-feet.*	*Acre-feet.*
0	0		
		625	625
25-foot	50		
		4,625	5,250
50-foot	320		
		11,875	17,125
75-foot	630		
		20,500	37,625
100-foot	1,010		
		31,375	69,000
125-foot	1,500		

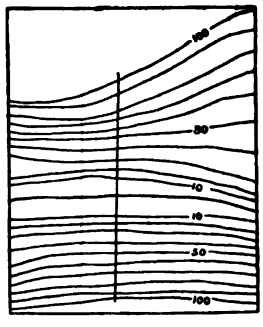


FIG. 4.—Map of Little Indian Valley dam site.

The only records of rainfall on the North Fork are those which have been kept for the last three seasons at Bartlett Springs. The precipitation for the season 1897–98 was 21.93 inches, which may be taken as the minimum. Rice's map of Lake County gives a drainage area of about 100 square miles above Little Indian Valley.

Using Mr. Lippincott's provisional curve for the run-off of central and southern California basins, the minimum precipitation of 22 inches should give a run-off of 16,000 acre-feet, and a precipitation of 38 inches—that estimated for this basin for a period of twenty-three years—should give a run-off of 58,000 acre-feet. To guard against a shortage in extremely dry years, however, a reserve of 29,000 acre-feet should be maintained during the years of average and excessive precipitation. On this basis an annual supply of 40,000 acre-feet would be available. The following table is based on the assumption that there were 29,000 acre-feet in the reservoir at the end of the irrigation season of 1885. Of the fourteen years considered, there would have been twelve years of full supply and two years of deficient supply. As experience shows that an occasional dry year may be tided over, it is believed that this site may be relied upon for 40,000 acre-feet.

A. LITTLE INDIAN VALLEY, LOOKING TOWARD DAM SITE.

B. LITTLE INDIAN VALLEY DAM SITE FROM ABOVE

Table showing fluctuations in volume stored in Little Indian Valley reservoir

Year.	Estimated rainfall.	Estimated run-off.	In reservoir at end of irrigation season.	Surplus.	Deficiency.
	Inches.	*Acre-feet.*	*Acre-feet.*	*Acre-feet.*	*Acre-feet.*
1886	26.40	24,800	13,000		
1887	24.35	20,700	Empty.		6,300
1888	34.40	44,400	4,400		
1889	49.20	124,400	29,000	59,800	
1890	41.60	73,200	29,000	33,200	
1891	32.70	39,750	18,750		
1892	44.00	84,000	29,000	23,750	
1893	35.70	48,950	29,000	8,950	
1894	53.00	134,000	29,000	94,000	
1895	43.50	81,750	29,000	41,750	
1896	49.40	113,000	29,000	73,000	
1897	24.12	20,240	9,000		
1898	17.35	7,525	Empty.		23,475
1899	47.67	102,350	29,000	33,350	
1900	37.38				

The canyon which leads from the valley (see Pl. VIII, *B*) is only 40 or 50 feet wide, and a shaly slate impregnated with quartz and some calcite outcrops on the bottom and on the sides to a height of 20 feet. At the point selected for the dam site the width between the canyon walls 100 feet above the creek bed is only 317 feet, and at 125 feet it is only 435 feet. On the west, immediately above the dam site, at a height of 600 feet, is a rocky point from which it is proposed to transfer the rock, by cable, to the dam. The elevation of the dam site is estimated to be 1,600 feet. Careful examination of the bed rock, with a diamond drill, should be made before the construction of the dam is begun.

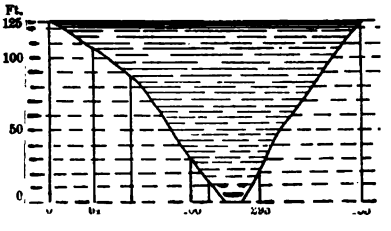


FIG. 5.—Elevation of Little Indian Valley dam site.

As the reservoir may be expected to fill during the wetter years, the dam must be an overflow weir. As designed, it will be 130 feet high, constructed of Cyclopean rubble, or of large blocks of stone set in concrete mortar, and adapted to withstand flotation effect with 10 feet of water passing over it.. The cost is estimated at $433,000. Allowing $20,000 for the purchase of the necessary territory in the valley, the total cost of the proposed Little Indian Valley reservoir is estimated at $453,000, which is at the rate of $6.56 per acre-foot of total storage capacity, or at the rate of $11.33 per acre-foot of water available.[1]

[1] The calculations of reservoir capacity were based on a dam 125 feet high. As the height of the weir dam designed is 130 feet, the available storage capacity is increased at least 7,500 acre-feet. Forty-seven thousand and five hundred acre-feet may therefore be considered as the available supply, which reduces the cost per acre-foot to $9.54.

CLEAR LAKE.

As has been stated, Clear Lake is situated on the Coast Range, 75 miles north of San Francisco. At mean low water the lake has a surface level of 1,325 feet above mean low water in San Francisco Bay. It is 20 miles long, 7 miles wide (greatest width), and has an area of 65 miles at its mean level. The greatest depth in the upper half of the lake is 35 feet, but in the lower portion there are small areas 50 feet deep. The drainage area is 417 square miles, and the storm waters are conducted to the lake through Scotts, Middle, and Clover creeks, at its extreme northwestern end, and by Doba, Kelsey, and Cole creeks on the southern side of its upper ends. These creeks are similar to others in Cache Creek Basin, in that they are torrential during the rainy season but are dry during the summer.

Middle and Clover creeks unite a mile below the town of Upper Lake. On July 12, 1900, their combined discharge was only 1.5 second-feet. Two miles above Clear Lake Scotts Creek flows into Tule Lake, a small, marshy depression, which it leaves through a slough. For 4 miles above Tule Lake the creek is simply a slough, and 6 miles above it had a discharge of 0.5 second-foot on July 12, 1900.

On July 16, 1900, Kelsey Creek had a discharge of 4.6 second-feet at a point 6 miles above the lake. A mile below, however, it had disappeared in its gravel bed. Doba and Cole creeks were both dry on that date. These three creeks flow through Big Valley, the largest and one of the most fertile in Lake County. Scotts Valley and the territory about Upper Lake are also remarkably productive, alfalfa being raised without irrigation. A good flow of artesian water is obtainable in the last-named districts at a depth of 75 feet, and it is now being used for irrigating vegetables.

In 1874–75 and in 1876–77 Clear Lake was one of the bodies of water examined by the engineers appointed to investigate the sources of water supply for the city of San Francisco. Recognizing the great loss due to evaporation over so extended an area, it was planned to "carry away the water from the lake in its season of abundance, and store it in a reservoir of less area and greater available depth. Such a reservoir exists on the adjoining stream to the south—Putah Creek, at Guenoc." This plan necessitated a tunnel 2.9 miles long. During these investigations (1874–1877) the water of Clear Lake was analyzed by four chemists. All were of the opinion that the water was entirely satisfactory for domestic purposes. On the next page is given the result of the analysis by Prof. Thomas Price.

A. LITTLE INDIAN VALLEY DAM SITE.

B. CANYON BELOW LITTLE INDIAN VALLEY DAM SITE.

Analysis of water from Clear Lake.

	Grains per gallon of 231 cubic inches.
Carbonate of lime	2.554
Carbonate of magnesia	2.683
Carbonate of soda	.728
Alumina and iron	.012
Chloride of potassium	.261
Sulphate of lime	.341
Silica	.464
Organic matter	1.970
Total	9.013

As water may contain as many as 80 grains of soluble salts before reaching the danger point for irrigation purposes, it is certain that Clear Lake water is satisfactory.

AREA AND VOLUME.

In 1889 the United States Geological Survey made a topographic survey of Clear Lake and its outlet. A datum plane was taken 1.27 feet above the low-water mark of 1873 and called 100. Mr. Wm. Ham. Hall compiled a number of tables for his report on this survey, the first four of which are given herewith.

The following table gives the area, in square miles, and the number of acres of each of the even-foot planes, from 90 to 110 feet—that is, 10 feet below and above the datum plane. The volume, in acre-feet, is also given for each 2-foot layer.

Area and volume of Clear Lake.

Elevation of plane.	Area of plane.	Area of plane.	Volume.
Feet.	*Square miles.*	*Acres.*	*Acre-feet.*
90	56.852	36,385	
			73,550
92	58.071	37,165	
			75,088
94	59.255	37,923	
			76,716
96	60.614	38,793	
			78,573
98	62.156	39,780	
			80,602
100	63.784	40,822	
			82,247
102	64.726	41,425	
			84,344
104	67.061	42,919	
			87,209
106	69.203	44,290	
			89,429
108	70.530	45,139	
			91,242
110	72.036	46,103	

The following table gives the equivalents of flow, in second-feet, corresponding to the several volumes in the successive 2-foot layers for 1 day, for 30 days, for 91 days, and for 182½ days, respectively:

Table of equivalents of flow.

Volume of layers.	Equivalents, in second-feet.			
	1 day.	30 days.	91 days.	182½ days.
Acre-feet.				
73,550	37,081	1,236	407	203
75,088	37,857	1,262	416	208
76,716	38,677	1,289	425	212
78,573	39,614	1,320	435	217
80,602	40,637	1,354	447	223
82,247	41,466	1,382	456	227
84,344	42,522	1,417	467	233
87,209	43,968	1,466	483	241
89,429	45,087	1,503	495	247
91,242	46,017	1,534	505	252

FLUCTUATIONS IN LEVEL.

In the following table is given the mean monthly and annual level of the surface of Clear Lake. The records up to 1888 were given to Mr. Hall by Captain Floyd, of Kono Tayee. The later records were obtained this summer (1900) from Mr. F. H. Porter, of Kono Tayee, and from Captain Rumsey and Captain Atherton, of Lakeport.

Table showing mean monthly and annual level of Clear Lake, 1874–1900.

Month.	1874.	1875.	1876.	1877.	1878.	1879.	1880.
January	102.87	102.96	102.73	102.04	102.42	.....	103.03
February	105.13	104.07	105.64	102.91	107.60	101.81	103.48
March	105.95	103.67	108.39	103.20	109.37	104.07	104.02
April	105.85	103.10	107.64	102.86	.....	105.73	106.19
May	104.99	102.40	106.23	102.28	.....	104.98	107.11
June	104.06	101.78	105.19	101.68	.....	.....	105.69
July	103.13	101.21	104.52	101.19	.....	103.13	104.56
August	102.31	100.55	104.04	100.71	.....	102.40	103.40
September	101.61	99.98	103.74	100.13	.....	101.90	102.51
October	101.23	99.58	103.16	99.62	.....	101.50	102.04
November	101.65	99.82	102.61	99.52	.....	101.24	101.64
December	102.04	100.75	102.02	99.56	.....	102.47	102.85
Mean	103.406	101.656	104.663	101.308	106.463	102.923	103.379

Table showing mean monthly and annual level of Clear Lake, 1874–1900—Cont'd.

Month.	1881.	1882.	1883.	1884.	1885.	1886.	1887.
January	104.88	101.24	100.37	99.13	102.43	105.16	.....
February	107.15	101.92	.....	99.68	102.73	106.14	101.75
March	106.88	103.11	100.77	100.83	102.54	105.35	102.89
April	105.75	103.71	101.23	101.88	102.23	108.30	.....
May	104.84	.....	101.58	103.01	101.79	105.19	102.50
June	104.16	.....	101.31	102.74	101.26	103.86	.....
July	103.08	.....	.....	102.20	100.81	102.73	.....
August	.....	.....	100.18	101.47	100.29	.....	100.77
September	101.26	100.27	99.73	100.98	99.65	.....	100.20
October	100.93	100.08	99.29	100.66	99.23	.....	.....
November	100.76	101.21	99.15	100.36	100.56	.....	.....
December	100.95	100.27	.....	101.19	102.87	100.83	99.48
Mean	103.695	101.476	100.401	101.178	101.366	104.32	101.265

Month.	1888.	1889.	1890.	1891.	1892.	1893.	1894.
January	99.83	.....	111.31	101.08	101.35	104.56	104.40
February	101.73	.....	.....	102.81	101.73	106.52	105.33
March	102.45	.....	.....	104.02	101.73	107.35	106.31
April	102.27	103.53	.....	104.09	102.15	.....	.....
May	101.96	.....	.....	103.52	102.33	104.73	.....
June	101.36	.....	.....	102.37	.....	103.46	.....
July	.....	.....	.....	101.56	.....	104.48	.....
August	100.08	.....	.....	100.81	.....	.....	.....
September	99.84	.....	101.86	.....	100.16	.....	.....
October	99.57	100.53	101.48	100.23	99.73	100.93	.....
November	99.34	.....	101.07	100.01	100.23	100.54	100.15
December	99.83	.....	100.97	100.56	104.47	101.40	.....
Mean	100.751	102.03	103.34	101.91	101.54	103.77	104.05

Month.	1895.	1896.	1897.	1898.	1899.	1900.	Monthly mean.
January	109.90	100.56	102.62	100.20	98.66	102.00	102.74
February	109.00	105.40	104.73	100.54	99.32	102.09	103.72
March	106.96	.....	105.63	101.00	99.45	102.94	104.12
April	105.73	104.98	105.52	100.73	100.40	102.92	103.81
May	.....	.....	104.35	100.40	99.84	102.29	103.32
June	.....	104.50	103.15	99.98	99.69	101.57	102.66
July	.....	103.46	102.32	99.46	99.42	100.82	102.24
August	.....	102.34	101.42	98.73	98.88	99.98	101.08
September	.....	101.36	100.73	98.20	98.25	99.40	101.24
October	.....	100.88	100.20	97.84	97.92	.....	100.32
November	100.23	100.59	99.96	97.75	98.21	.....	100.30
December	.....	101.32	100.09	97.86	99.57	.....	100.92
Mean	106.36	102.54	102.56	99.39	99.13	101.55	*a* 102.51

a Twenty-six years.

The table on the next page gives the extreme high-water levels and the extreme low-water levels of the lake for each year from 1873 to 1900 (compiled from the same records as the foregoing table), while fig. 6 shows graphically the same data. The number of feet fall in the lake during the year and the evaporation for the falling period are also

The following table gives the equivalents of flow, in second-feet, corresponding to the several volumes in the successive 2-foot layers for 1 day, for 30 days, for 91 days, and for 182½ days, respectively:

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March	105.95	103.67	108.39	103.20	109.37	104.07	104.02
April	105.85	103.10	107.64	102.86	.....	105.73	106.19
May	104.99	102.40	106.23	102.28	.....	104.98	107.11
June	104.06	101.78	105.19	101.68	.....	.....	105.62
July	103.13	101.21	104.52	101.19	.....	103.13	104.56
August	102.31	100.55	104.04	100.71	.....	102.40	103.40
September	101.63	99.98	103.74	100.13	.....	101.90	102.51
October	101.23	99.58	103.16	99.62	.....	101.50	102.04
November	101.65	99.82	102.61	99.52	.....	101.24	101.64
December	102.04	100.75	102.02	99.56	.....	102.47	102.85
Mean	103.406	101.656	104.663	101.308	106.463	102.923	103.879

Table showing mean monthly and annual level of Clear Lake, 1874–1900—Cont'd.

Month.	1881.	1882.	1883.	1884.	1885.	1886.	1887.
January	104.88	101.24	100.37	99.13	102.43	105.16	.....
February	107.15	101.92	.....	99.68	102.73	106.14	101.75
March	106.88	103.11	100.77	100.83	102.54	105.35	102.89
April	105.75	103.71	101.23	101.88	102.23	108.30	.....
May	104.84	.....	101.58	103.01	101.79	105.19	102.50
June	104.16	.....	101.31	102.74	101.26	103.86	.....
July	103.08	.....	.....	102.20	100.81	102.73	.....
August	.....	.....	100.18	101.47	100.29	.....	100.77
September	101.26	100.27	99.73	100.98	99.65	.....	100.20
October	100.93	100.08	99.29	100.66	99.23	.....	.....
November	100.76	101.21	99.15	100.36	100.56	.....	.....
December	100.95	100.27	.....	101.19	102.87	100.83	99.48
Mean	103.695	101.476	100.401	101.178	101.366	104.32	101.265

Month.	1888.	1889.	1890.	1891.	1892.	1893.	1894.
January	99.83	.....	111.31	101.08	101.35	104.56	104.40
February	101.73	.....	.....	102.81	101.73	106.52	105.33
March	102.45	.....	.....	104.02	101.73	107.35	106.31
April	102.27	103.53	.....	104.09	102.15	.....	.....
May	101.96	.....	.....	103.52	102.33	104.73	.....
June	101.36	.....	.....	102.37	.....	103.46	.....
July	.....	.....	.....	101.56	.....	104.48	.....
August	100.08	.....	.....	100.81	.....	.....	.....
September	99.84	.....	101.86	.....	100.16	.....	.....
October	99.57	100.53	101.48	100.23	99.73	100.93	.....
November	99.34	.....	101.07	100.01	100.23	100.54	100.15
December	99.83	.....	100.97	100.56	104.47	101.40	.....
Mean	100.751	102.03	103.34	101.91	101.54	103.77	104.05

Month.	1895.	1896.	1897.	1898.	1899.	1900.	Monthly mean.
January	109.90	100.56	102.62	100.20	98.66	102.00	102.74
February	109.00	105.40	104.73	100.54	99.32	102.09	103.72
March	106.96	.....	105.63	101.00	99.45	102.94	104.12
April	105.73	104.98	105.52	100.73	100.40	102.92	103.81
May	.....	.....	104.35	100.40	99.84	102.29	103.32
June	.....	104.50	103.15	99.98	99.69	101.57	102.66
July	.....	103.46	102.32	99.46	99.42	100.82	102.24
August	.....	102.34	101.42	98.73	98.88	99.98	101.08
September	.....	101.36	100.73	98.20	98.25	99.40	101.24
October	.....	100.88	100.20	97.84	97.92	.....	100.32
November	100.23	100.59	99.96	97.75	98.21	.....	100.30
December	.....	101.32	100.09	97.86	99.57	.....	100.92
Mean	106.36	102.54	102.56	99.39	99.13	101.55	*a* 102.51

a Twenty-six years.

The table on the next page gives the extreme high-water levels and the extreme low-water levels of the lake for each year from 1873 to 1900 (compiled from the same records as the foregoing table), while fig. 6 shows graphically the same data. The number of feet fall in the lake during the year and the evaporation for the falling period are also

For this purpose a provisional curve of run-off compiled by J. B. Lippincott from the data for a great number of California basins was used, but it was discovered that the results in many cases were less than the amounts shown to have entered the lake by the rise from extreme low water to extreme high water.

An inspection of the following table will show that the total amount calculated from Mr. Lippincott's provisional curve is too small in all cases where the rainfall is less than 19 inches and also in a number of other cases. Clearly, the so-called "intake" (the difference between extreme low water and extreme high water) is less than the volume which entered the lake, because it does not show either the discharge or the loss by evaporation during the time the level was rising.

Table showing fluctuations in volume of Clear Lake.

Season.	Rainfall.	Run-off from 413 square miles.	Rainfall on lake.	Total increase in volume.	Intake.
	Inches.	*Acre-feet.*	*Acre-feet.*	*Acre-feet.*	*Acre-feet.*
1873-74	14.6	16,930	49,080	66,010	279,300
1874-75	24.96	90,860	86,320	177,180	136,370
1875-76	37.0	223,020	128,650	351,670	458,650
1876-77	14.0	14,460	49,600	64,060	79,500
1877-78	54.0	586,660	186,750	773,410	458,650
1878-79	21.3	60,298	74,700	134,998	269,920
1879-80	29.0	115,640	102,960	218,700	362,720
1880-81	21.8	38,820	77,220	116,040	275,170
1881-82	16.7	28,910	58,100	87,010	127,570
1882-83	19.0	42,126	65,280	107,406	73,020
1883-84	24.3	93,340	81,600	174,940	164,900
1884-85	16.0	22,720	51,000	73,720	190,210
1885-86	35.5	198,240	124,500	322,740	316,900
1886-87	19.25	43,780	86,400	130,180	95,736
1887-88	17.88	35,100	61,350	96,450	123,480
1888-89	27.28	109,860	94,070	203,930	136,620
1889-90	51.43	536,900	182,300	719,200	474,360
1890-91	21.06	57,820	117,100	174,920	133,940
1891-92	24.32	85,080	83,000	168,080	111,700
1892-93	36.80	218,890	127,400	346,290	340,580
1893-94	32.82	165,200	112,050	277,250	248,170
1894-95	42.89	326,270	158,730	485,000	421,680
1895-96	33.48	169,330	116,200	285,530	219,410
1896-97	28.92	123,900	102,960	226,860	220,400
1897-98	14.57	16,930	48,960	65,890	89,570
1898-99	20.86	56,990	67,660	124,650	121,170
1899-1900	27.73	113,160	94,070	207,230	221,600
Total				6,179,344	6,151,296

Neglecting the winter discharge and the winter evaporation, and considering the totals given at the bottom of the last two columns in the foregoing table, the run-off shown by Mr. Lippincott's provisional curve agrees with that indicated by the rise of the lake within one-half of 1 per cent. Thus the curve is shown to give an excellent conservative estimate for average conditions, and for that reason it was used in determining the run-off of Little Indian Valley.

The following table shows the method adopted to determine the limits for the hold-over reservoir:

Table showing fluctuations in Clear Lake.

Season.	Intake.	Resulting level.	Winter discharge.	Summer evaporation.	Working level.	Depth used.	Fall level.
	Acre-feet.	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*	*Feet.*
1873 (low water)	.....	.....	.....	.....	.....	.....	98.7
1873–74	279,300	105.4	1.63	3.58	102.4	1.9	100.5
1874–75	136,370	103.8	0.76	4.09	100.5	2.0	98.5
1875–76	458,650	106.0	4.47	4.17	101.8	2.0	99.8
1876–77	79,500	101.7	0.29	3.58	98.4	2.0	96.4
1877–78	458,650	106.0	6.27	4.17	101.8	2.0	99.8
1878–79	269,920	106.0	0.84	3.91	102.1	1.9	100.2
1879–80	280,440	103.0	2.58	3.67	102.3	1.9	100.4
1880–81	275,170	106.0	3.08	4.09	101.9	2.0	99.9
1881–82	129,370	103.0	.....	3.77	99.2	2.0	97.2
1882–83	74,021	99.1	.....	2.65	96.5	2.0	94.5
1883–84	169,120	98.8	0.33	3.84	95.3	2.1	93.2
1884–85	190,210	98.1	0.74	3.87	95.0	2.1	92.9
1885–86	316,900	101.0	1.43	4.38	98.0	2.0	96.0
1886–87	95,740	98.4	.....	3.60	94.8	2.1	92.7
1887–88	123,480	95.9	.....	3.30	92.6	2.2	90.4
1888–89	177,740	95.2	.....	3.00	92.2	2.2	90.0
1889–90	474,360	106.0	6.21	4.20	101.8	2.0	99.8
1890–91	133,940	103.0	0.45	3.67	99.7	2.0	97.7
1891–92	111,700	100.5	.....	3.33	97.2	2.0	95.2
1892–93	340,580	103.6	2.96	3.91	102.1	1.9	100.2
1893–94	246,170	106.0	2.25	3.91	102.1	1.9	100.2
1894–95	421,680	a 109.9	5.47	4.20	101.8	2.0	99.8
1895–96	219,410	105.0	0.75	4.09	101.7	2.0	99.7
1896–97	220,400	104.9	2.24	3.67	102.3	1.9	100.4
1897–98	49,260	101.6	.....	3.33	98.3	2.0	96.3
1898–99	121,170	99.3	.....	2.83	96.5	2.0	94.5
1899–1900	221,601	100.1	.....	.....	.....	.....	.....

a Maximum level.

Referring to the foregoing table, the "intake" is the increase in volume represented by the difference between the low-water and the high-water levels. By the "resulting level" is meant the height the lake's surface would attain after the addition of the intake to the previous low-water level. Thus, 279,300 acre-feet flowing into the lake when the level was 98.7 feet (the low-water mark of 1873) would cause a rise to 105.4 feet. The "summer evaporation" is taken from the table on page 36. The "winter discharge" is based on the assumption that as much water is discharged while the lake is rising as is discharged while it is falling through the same distance. As the outflow in the later fall months is very small, the periods of discharge may be taken as approximately equal and the element of time be eliminated. If there be any fluctuations in level while the lake is rising, which is far more probable than while it is falling, then the "winter discharge" must exceed the "summer discharge;" so whatever error there may be in the above assumption is on the side of conservatism. The "winter discharge" given in this table is, therefore, the "summer discharge" found by subtracting the evaporation given in the table on page 36 from the fall of the lake's surface given in the same table. In the cases where the evaporation is greater than the fall, nothing is allowed for the "winter discharge." The "working level" is found by subtracting the difference between the "winter discharge" and the "summer evaporation" from the "resulting level;" it represents the level at which the lake would stand in the fall were no water taken out for irrigation. As the top of the outlet weir is fixed at 106 feet, whenever the lake rises above that level the water is wasted. Accordingly, whenever the "resulting level," or the "resulting level plus the "winter discharge," is above 106 feet, the "summer evaporation" is subtracted from 106 feet, as it is at that point that the evaporation begins. The "depth used" shows the depth of reservoir necessary to store 80,000 acre-feet of water at the various levels. Subtracting the "depth used" from the "working level," the minimum "fall level" is obtained.

In the season of 1888–89, under our hypothetical conditions the lake would have fallen to 90 feet, and in the previous year to 90.4 feet. The next lowest levels would have been 92.7 feet and 92.9 feet, in the seasons of 1886–87 and 1884–85, respectively. Following our basis of calculation, to allow 80,000 acre-feet for each and every irrigating season the outlet of the lake should be fixed at 90 feet. As no "winter discharge" has been allowed for a number of seasons, and, furthermore, as it seems that the rate of evaporation used is too large, it appears almost certain that 100,000 acre-feet might have been used continuously for the last twenty-seven years. This is tantamount to claiming that with the weir height and outlet fixed at 106 feet and 90 feet, respectively, 100,000 acre-feet may safely be drawn from Clear Lake reservoir every season.

trolling
of lake.
104
106
108
110
112
114
116
118
120
T.12N.

Cost of outlet channel.

Dredging (10 per cent of excavation), 201,336 cubic yards at $0.35	$70,468
Cartage (10 per cent of excavation), 201,337 cubic yards at $0.17	34,227
Excavation by wheel scrapers (80 per cent), 1,610,692 cubic yards at $0.10	161,069
Total	265,764

Referring again to the table on page 39, it will be seen that the greatest height which the water would attain was in 1894–95, when, if the whole amount had come from a single storm, the maximum level would have been 115.37 feet (109.9+5.47). As such a condition is not to be expected, the top of the embankment may safely be placed at

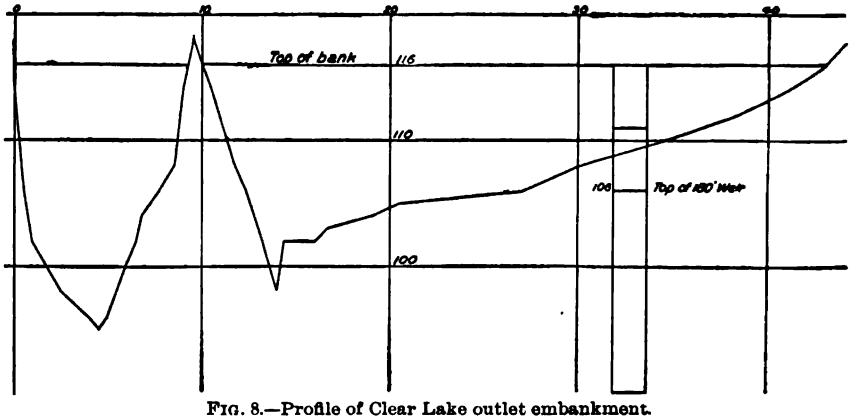


FIG. 8.—Profile of Clear Lake outlet embankment.

116 feet (10 feet above the level of the lake when full), which will allow a good margin for wave action. A profile of the embankment is shown in fig. 8. Following is the estimate on the embankment:

Embankment for Clear Lake reservoir.

Length.	Top width.	Inner slope.	Outer slope.	Elevation of top.	Filling.
Feet.	*Feet.*			*Feet.*	*Cu. yds.*
4,300	12	3 to 1	2 to 1	116	[a] 66,878

[a] Includes 10 per cent for shrinkage.

The cost of the embankment would be $16,720—66,878 cubic yards of filling at 25 cents.

The outlet weir may safely be estimated at $50,000. It is based on a design for 18 concrete piers, with openings 4 feet wide and 16 feet deep, closed by specially designed iron-bound flashboards working in iron grooves in masonry. The entire structure is to rest on a grillage of timber 40 feet by 180 feet by 2 feet thick, the whole supported

by 1,224 piles, driven at least 16 feet into the ground. A view of the regulating weir is shown in Pl. X.

The estimate also provides for heavy retaining walls of concrete resting on timber platforms on pile foundations.

The entire cost of the proposed works is as follows:

Cost of proposed storage works at Clear Lake.

Outlet channel	$265,764
Embankment	16,720
Riprapping	50,000
Outlet weir	50,000
Damages to property	70,000
Total cost	452,484

Considering 80,000 acre-feet as the maximum storage capacity, the cost becomes $5.66 per acre-foot; but if we count upon 100,000 acre-feet (not an improbable figure) as the maximum storage capacity, the cost is only $4.52 per acre-foot.

These matters are more fully discussed in the appended report on the Clear Lake outlet, by Mr. J. H. Quinton.

LANDS FOR RESERVOIR.

In the Eleventh Annual Report of the United States Geological Survey, Part II (Irrigation), pages 159 to 164, there is given a list of the segregations suggested in 1889 for the Clear Lake reservoir. Pl. LXXXV, page 151 of that report, is a map of this territory, the aggregate of which is 50,920 acres. Virtually all of the land recommended for segregation was taken up prior to 1889. The land on the immediate shore of the lake was necessarily sold in lots, the outer boundary being the meander shore line of the land survey. With the exception of small tracts taken up in 1890 and 1894, none of the land listed has been taken up since 1888. The purchase of 1890 is as follows:

SW. ¼ of NW. ¼ of sec. 20, T. 14 N., R. 8 W., M. D. M.

The land purchased in 1894 was taken up as California swamp and overflow land, and is as follows:

Lots 1, 2, 3, 4, and 5 (113 acres) of sec. 28, T. 14 N., R. 9 W., M. D. M.
NW. ¼ of NW. ¼ of sec. 33, T. 14 N., R. 9 W., M. D. M.

The following is a list of the remaining territory:

SE. ¼ of NE. ¼ of sec. 32, T. 13 N., R. 8 W., M. D. M.
S. ½ of NW. ¼ of sec. 36, T. 14 N., R. 8 W., M. D. M.
S. ½ of NE. ¼ of sec. 36, T. 14 N., R. 8 W., M. D. M.
NW. ¼ of sec. 36, T. 14 N., R. 9 W., M. D. M.
NE. ¼ of NE. ¼ of sec. 36, T. 14 N., R. 10 W., M. D. M.
W. ½ of SE. ¼ of sec. 36, T. 15 N., R. 9 W., M. D. M.
SE. ¼ of SW. ¼ of sec. 36, T. 15 N., R. 9 W., M. D. M.

Thus, out of the 50,920 acres recommended for segregation, 193 acres have been taken up since 1889, the date of the recommendation, and 520 acres still remain as public land. In considering this remainder, 520 acres, no account has been taken of the land which lies within the meander shore line.

CLEAR LAKE OUTLET.[1]

In designing a regulating weir for the outlet of Clear Lake, Lake County, California, the following conditions had to be observed:

(1) To regulate levels of water between elevations of 90 and 106 feet above datum.

(2) To provide for measuring accurately the quantity of water to be delivered at any and all times during the irrigating season.

(3) To provide sufficient waterway over the weir, with all openings closed, to carry the greatest possible flood at full reservoir without raising the water to such a height as to inundate valuable lands adjacent to the lake.

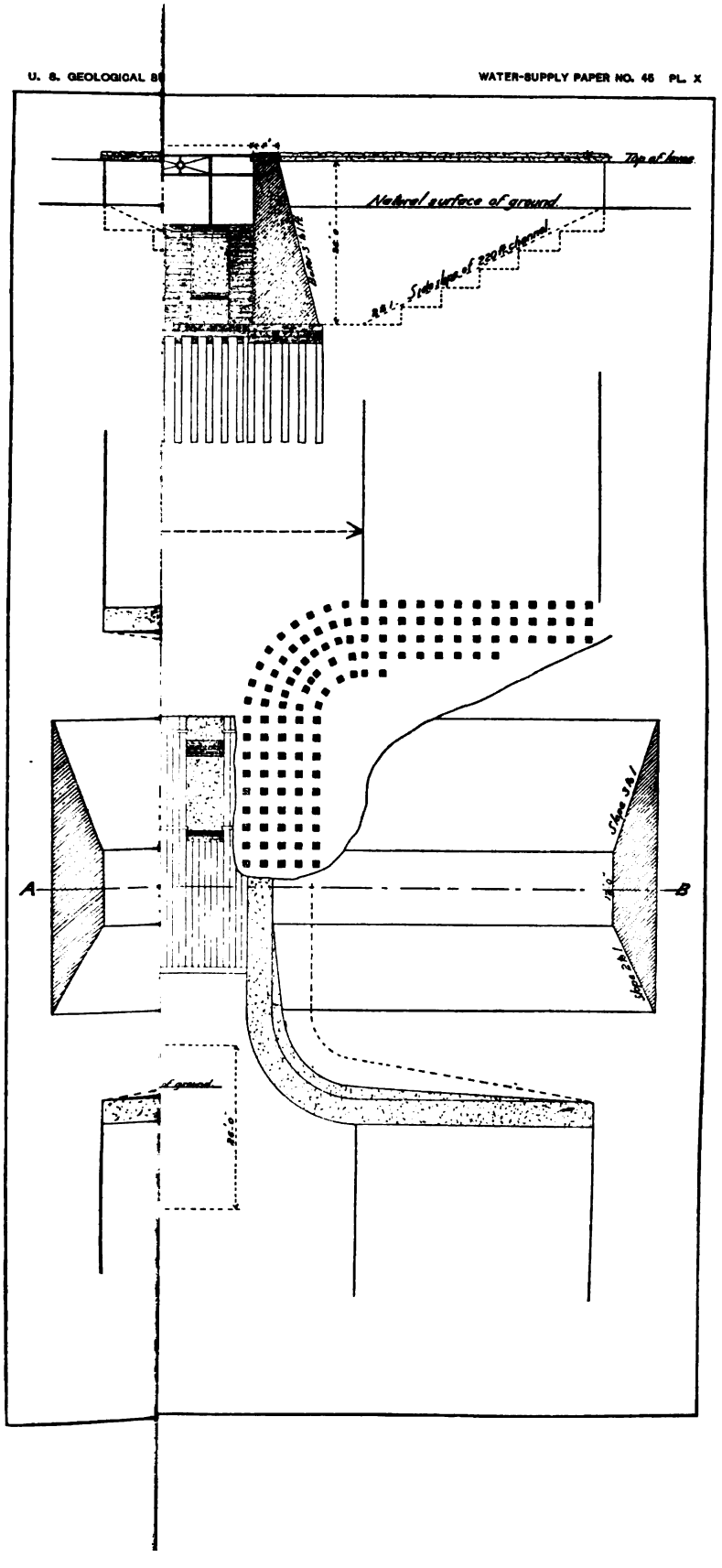
(4) To arrange the outlets so as to give the maximum delivery for minimum cost of construction and maintenance.

(5) To make gates so simple as to preclude their getting out of order.

As designed (see Pl. X), the weir consists of 18 piers of concrete, each 6 feet wide, 16 feet high, and 20 feet thick at base, with 19 openings, each 4 feet wide and 16 feet deep, closed by flashboards working in steel-lined grooves in the sides of the piers, the whole supported by a double grillage of 12-inch by 12-inch timbers resting on more than 1,200 piles. The ends of the weir are formed of heavy curved retaining walls of concrete, as shown in Pl. X. The curved portions of the retaining walls to be strengthened for tensile strains from the pressure of earth against the straight walls when the water is low, by embedding old steel cables or rods in the concrete around the curves and extending them into the straight walls at each end. The flashboards are 12 inches wide and 4 inches thick, and are bound with steel on the ends and sides, so as to fit accurately when dropped into place.

Through each flashboard two steel rods will be secured, so as to project about 1½ inches on each side of the board, to facilitate raising it when submerged. Suitable hooks attached to light chains can be dropped under these projecting rods, and in case of much pressure a small portable tripod with differential pulley attached may be set up on the footbridge, to start the flashboard from its seat. The footbridge (designed to afford access to the flashboards when the water is high) is supported on trestlework running across the top of the weir. The bents of this trestle are 10 feet apart, and are anchored in the tops of piers. When the water is low, the foot planks can be supported

[1] Report of J. H. Quinton.


Top of dam
Natural surface of ground
Slope 3 to 1
Slope 2 to 1
A
B

on steel rods projecting from the upper sides of the piers. When the level of the water is at the top of the weir, the following amounts would flow through the openings:

Discharge through weir openings.

Number of flashboards removed.	Discharge through 1 opening.	Discharge through 19 openings.
	Cu. ft. per second.	*Cu. ft. per second.*
1	13.3	252.7
2	37.3	708.7
3	69.2	1,314.8
4	106.6	2,025.4
5	149.2	2,834.8

Assuming the duty of the reservoir to be 80,000 acre-feet annually, and that this amount will have to be delivered through the openings in 120 days (the length of the irrigating season), the amount of water which would have to be turned out daily during that time would be—

$$\frac{80{,}000 \text{ acre-feet}}{2 \times 120} = 333 \text{ second-feet.}$$

So that, except in case of a storm, it would seldom be necessary to remove more than two flashboards the entire length of the weir.

When the water is level with the top of the weir, the water pressure on the boards would be as follows:

First board, $4 \times 31.25 = 125$ pounds.
Second board, $4 \times 1.5 \times 62.5 = 375$ pounds.
Third board, $4 \times 2.5 \times 62.5 = 625$ pounds.

As the coefficient of friction for steel on steel is only 0.14, the pull required to move the third board would be only 0.14×625 pounds= 87.5 pounds, and this could easily be exerted without the aid of a pulley.

When the water is at the level of the top of the weir, the greatest pressure on the lowest flashboard is 4,000 pounds. Taken as a beam supporting this uniform load, the skin stress on the lumber would be only 750 pounds per square inch, a factor of safety of at least 7 for Oregon pine. Under similar circumstances the greatest pressure on a pier, including that on the flashboards on each side, would be $10 \times 16 \times 8 \times 62.5 = 80{,}000$ pounds, acting horizontally at one-third of the height of the pier from the base. The concrete in the pier measures 1,320 cubic feet and weighs approximately $1{,}320 \times 130 = 171{,}600$ pounds. With three flashboards removed from each opening and with 2,835 second-feet of water flowing into the lower channel, the depth of water here would be 6 feet; and if the water worked its way

underneath, the pier might lose in weight about 45,000 pounds. When all of the flashboards are in place and 5 feet depth of water (6,832 second-feet) is flowing over the weir, the depth of water in the channel below would be 10 feet. Of course this would be an extreme case and one quite unlikely to occur if the flashboards were removed as the water rose, so as to keep the water as nearly at the level of the top of the weir as possible. In such a case, however, the piers would practically be submerged and would lose considerable of their weight, so that the possibility of their being moved bodily downstream had to be considered. To prevent this tendency, the piers are built on the first platform of 12-inch by 12-inch timbers, which is bolted to the caps on the piles. The second or upper platform of 12-inch by 12-inch timbers, which is securely bolted to the first platform, surrounds the piers on three sides, so that there is no possibility of the piers being moved downstream.

To prevent too much vibration from the water falling over the flashboards, secondary steel grooves are placed near the lower end of the piers, in which three or four flashboards may be dropped, so as to form a cushion for the falling water.

In order to keep the platform constantly wet during low water, which is essential to its preservation, a 12-inch by 12-inch timber is run the entire distance across its lower edge, being securely bolted to it with driftbolts. This forms a second cushion for the water flowing from the openings in the weirs, and also keeps the entire platform and piling continually submerged.

The measurement of any quantity of water during the irrigation season can easily be effected—(1) by removing the number of boards necessary to measure quantities greater than 13 cubic feet per second, and (2) by raising one flashboard, so as to form a submerged opening of the size required, to measure smaller quantities. It is evident that within reasonable limits this will afford a very effective and simple method of measurement.

It is proposed to run small horizontal grooves a few inches apart on the sides of the piers, so as to admit air under the jets falling over the flashboards.

It is believed that the weir as designed will meet all requirements.

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56TH CONGRESS, 2d Session. } HOUSE OF REPRESENTATIVES. { DOCUMENT No. 530.

DEPARTMENT OF THE INTERIOR

WATER-SUPPLY

AND

IRRIGATION PAPERS

OF THE

UNITED STATES GEOLOGICAL SURVEY

No. 46

RECONNAISSANCES OF KERN AND YUBA RIVERS, CALIFORNIA.—
OLMSTED AND MANSON

WASHINGTON
GOVERNMENT PRINTING OFFICE
1901

UNITED STATES GEOLOGICAL SURVEY

CHARLES D. WALCOTT, DIRECTOR

PHYSICAL CHARACTERISTICS OF KERN RIVER, CALIFORNIA

BY

FRANK H OLMSTED

AND

RECONNAISSANCE OF YUBA RIVER, CALIFORNIA

BY

MARSDEN MANSON

WASHINGTON

GOVERNMENT PRINTING OFFICE

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LETTER OF TRANSMITTAL.

DEPARTMENT OF THE INTERIOR,
UNITED STATES GEOLOGICAL SURVEY,
DIVISION OF HYDROGRAPHY,
Washington, D. C., March 6, 1901.

SIR: I have the honor to transmit herewith two manuscripts for publication together in the series of papers upon water supply and irrigation.

The first manuscript is by Mr. Frank H. Olmsted, a civil engineer of southern California, and relates to the physical characteristics of the catchment area of Kern River, California. In this paper the author describes the drainage basin and estimates the amount of water coming from it and the power available from a complete utilization of the various portions of the stream. He also discusses the utilization of this power in pumping for irrigation.

The ultimate development of a considerable portion of the arid region depends not only upon complete storage and control of the streams, but upon the largest possible use of the power which may be generated in the upper or steeper portions of the channel and transmitted electrically out upon the plains, to be used in bringing to the surface the moisture which has sunk below the reach of the roots of the plants. By this means the area of cultivation can be greatly extended; and, as shown by Mr. Olmsted, the cost of pumping this water and applying it to the soil may, under favorable conditions, be less than that of obtaining a supply by gravity.

In earlier pamphlets of this series of Water-Supply Papers the utilization of wind power for this purpose has been discussed, the efficiency of the windmill has been described, and the advantages due to the small cost and independent construction of each mill have been shown. Where, however, it is practicable to obtain electric power at small cost, pumping plants operated by electricity can be widely distributed and may have certain advantages over the windmill. It is therefore important, in any discussion of the method of utilizing the water resources, to bring to public attention the possible developments along this line.

The second manuscript presented herewith has been prepared by Dr. Marsden Manson, and relates to Yuba River, a tributary of the Sacramento. Dr. Manson discusses the physical conditions and storage possibilities of this stream, bringing out particularly the importance of preserving the forest cover on the upper catchment basin,

and, if possible, increasing this by artificial means; and shows by estimates the possible increase of available water through complete afforestation of the area. The relation of forests to river flow is believed to be of great importance, and is a matter upon which precise data are needed. It is hoped that as the systematic river measurements continue it will be possible to state more and more definitely the precise relation which the forest cover bears to the behavior of the stream.

The protection of the forests by the creation of reservations and the conservation of the waters through reservoirs constructed within these various reservations are of such vital importance to the utilization of the arid lands of the West, both by direct irrigation and by the creation of power for pumping water, that all matters pertaining to these subjects have interest to the citizens of the country as the great landowners.

It is for these reasons that these manuscripts are presented as contributions to a larger knowledge of the subject.

Very respectfully,

F. H. NEWELL,
Hydrographer in Charge.

Hon. CHARLES D. WALCOTT,
Director United States Geological Survey.

PHYSICAL CHARACTERISTICS OF KERN RIVER, CALIFORNIA, WITH SPECIAL REFERENCE TO ELECTRIC POWER DEVELOPMENT.

By FRANK H. OLMSTED.

INTRODUCTION.

The development of southern California has been retarded by three factors which are still operative: (1) Distance from the world's markets and commercial centers; (2) shortage of water; (3) lack of cheap fuel and power. Volumes might be written in regard to the relation which the development of water in southern California bears to the progress of the people, but broadly stated it may be said that the real development of this section will be defined and limited by the amount of water available for irrigation from San Diego to Santa Barbara and from San Pedro to the desert on the east. Regarding cheap fuel and power, neither Los Angeles, the commercial center of this section, nor any other place in southern California has had it, and even now, when it is reasonable to suppose that the local oil industry is at its best, the price of oil is approaching $1 a barrel of 42 gallons; or, expressed differently, the cost of oil as fuel is equal to bituminous coal at $4 a ton. During the last five years Los Angeles has had an industrial awakening corresponding to this decrease in the cost of fuel from $7 or $8 a ton to $4 a ton.

During the first year of its advent in the Los Angeles market the cost of oil ranged between $0.40 and $1 a barrel. Most of the users of coal at that time found it economical to change their grates to oil burners, but since 1896 the tendency of the oil market has been upward, until now the cost of oil is $0.75 to $1 a barrel. At $1.50 a barrel oil as fuel is no cheaper than Gallup coal at $7 a ton, except that usually the oil feeder is arranged so as not to require an attendant, thus saving the wages of a stoker. In carload lots, the cost of Wellington bituminous coals is about $1 a ton more than the Eastern lignites; and, limited as the output has been, at present it virtually

controls the Los Angeles market. During at least half of the year the demand in Los Angeles is so near the greatest available supply that outside orders for coal are not sought. The oil supply has contributed to the industrial advancement of southern California, and there are many reasons for believing that with cheap electric power

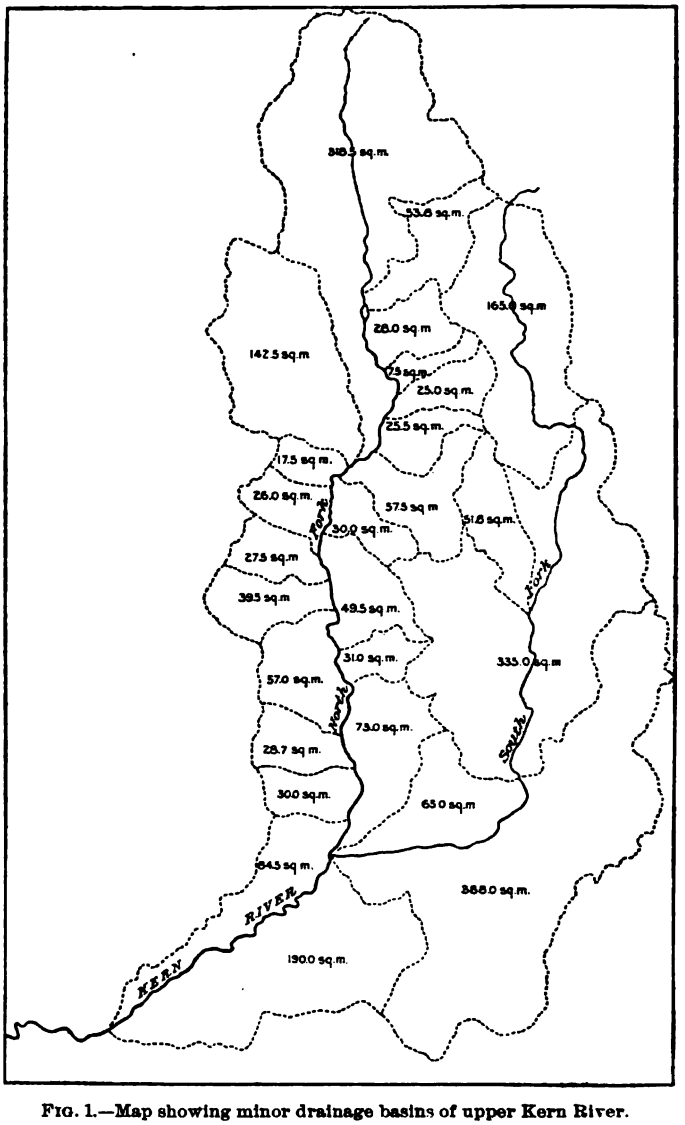


FIG. 1.—Map showing minor drainage basins of upper Kern River.

the manufactures and industries of southern California in general, and of the city of Los Angeles in particular, would quickly respond, and that their growth would be upon a safer foundation and a broader basis. The future outlook does not, however, warrant the hope for much better fuel rates, even with the construction of the

Salt Lake Railroad assured; but there is good reason to expect cheaper power, and if this expectation is realized, the greater part of the power must come from Kern River.

PHYSICAL FEATURES.

Kern River rises on the western slope of the Sierra Nevada, the greatest mountain range in the United States. For a distance of 100 miles the average elevation of this crest line is more than 11,000 feet above the sea, and so great is the run-off that within the first 15 miles of its course the river receives 80 per cent of its total summer flow at the mouth of its canyon 100 miles away. The drainage area of the stream above the latter point is 2,349.3 square miles. Fig. 1 is a map showing the distribution of this drainage area, the minor drainage basins being outlined and their respective areas given. The following tables give the names and the respective areas of these minor drainage basins:

Areas of minor drainage basins of North Fork of Kern River.

	Square miles.
Headwaters	318.5
Whitney Creek	53.8
Small Creek	28.0
Ninemile Creek	7.5
Menache Creek	25.0
Trout Meadow Creek	25.5
Harris Creek	57.5
Tibbetts Creek	30.0
Brush Creek	49.5
Salmon Creek	31.0
Corral and other creeks	75.0
Little Kern River	142.5
Needles Creek	17.5
Clark Creek	26.0
Jackson Creek	27.5
Wade Creek	39.5
Tobias and other creeks	57.0
Bull Run Creek	28.7
Tilly Creek	30.0
Total to junction of South Fork	1,070.0

Areas of minor drainage basins of South Fork of Kern River.

	Square miles.
Headwaters	165.0
Fish Creek	51.8
Middle tributaries	335.0
Lower tributaries from north	65.0
Lower tributaries from south	388.0
Total to junction with North Fork	1,004.8

Drainage areas of Kern River.

	Square miles.
North Fork	1,070.0
South Fork	1,004.8
Tributaries from north side after junction	84.5
Tributaries from south side after junction	190.0
Total	2,349.3

The length of Kern River from King River summit, on its main fork, to the mouth of the canyon above Bakersfield is 118 miles. The channel is in granite, and, with the exception of a few drops in the lower reaches of the stream, the grades are fairly uniform. In

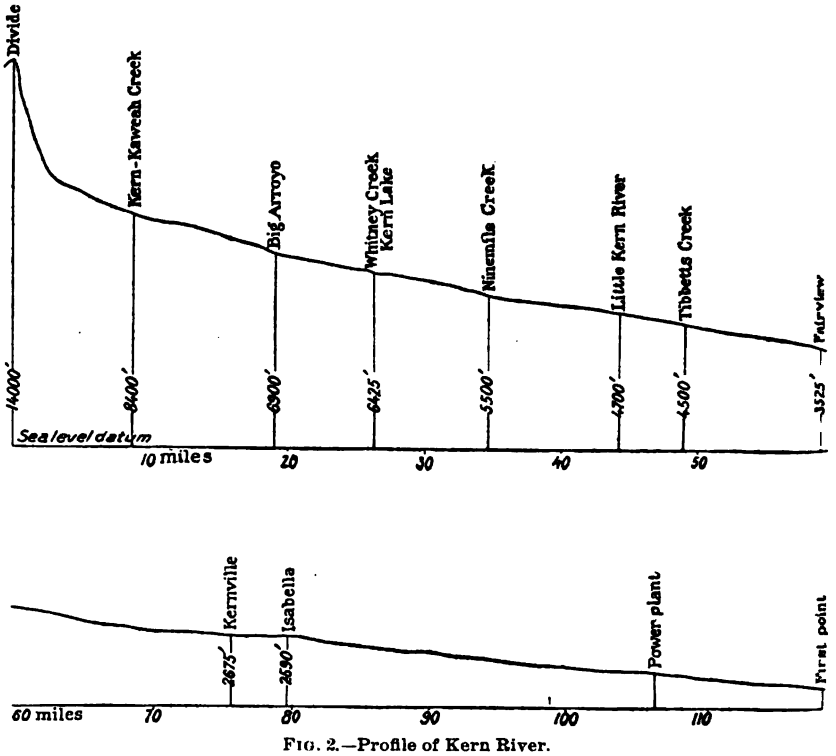


FIG. 2.—Profile of Kern River.

the 62 miles above Kernville the stream falls 5,600 feet, and in the 48 miles below Kernville it falls about 2,100 feet. A view of the river near Kernville is shown in Pl. I, and a profile of the stream in fig. 2. Fig. 3 is a view of a mountain valley in the basin of the river.

The South Fork of the stream, which rises just south of Cirque Peak, is 83 miles long to its junction with the North (or main) Fork at Isabella. The main range of the Sierra Nevada drops off rapidly just south of Cirque Peak, which may account in a large measure for the comparatively small flow of the South Fork. This branch of Kern River possesses one decided advantage over the other branch,

VIEW OF KERN RIVER NEAR KERNVILLE.

and over most California streams, in that it has a succession of particularly fine reservoir sites along a channel otherwise distinguished by reaches of rapid descent. A profile of the South Fork is shown in fig. 4.

The entire flow of Kern River is utilized for irrigation in the

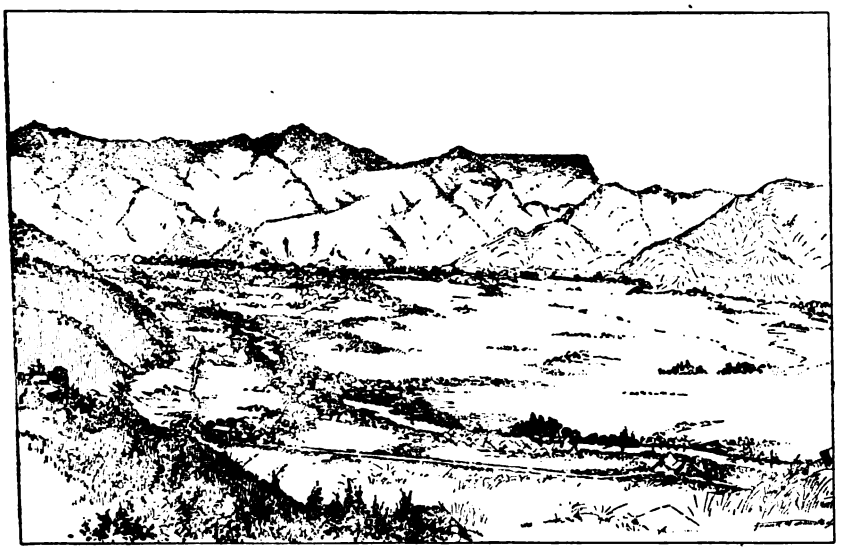


FIG. 3.—View of mountain valley in Kern River Basin.

southern end of San Joaquin Valley, and were it not for the great evaporation losses from the Buena Vista Lake reservoir the system could be called effective. The power of the stream, which, with one small exception, is available in large units, wastes itself upon the marbled bowlders and granite bed rock of the canyon, and suggests

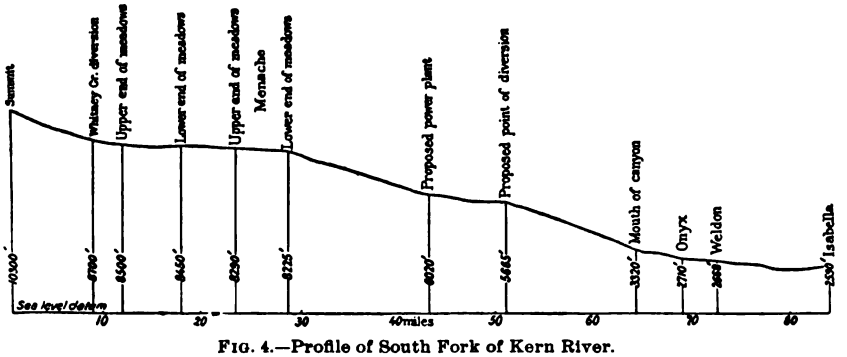


FIG. 4.—Profile of South Fork of Kern River.

an encouraging field for immediate investigation. The need of power in the agricultural valleys of California was never so apparent as now, after a series of dry years, when the ordinary flow of the streams has diminished to the extent of eliminating much hitherto valuable land, which of course precludes further development along

gravity lines. Power applied to the pumping of water from the gravel beds which underlie most of the California streams where they debouch from their canyons at once solves the difficulty; and where the water plane is not far below the surface this method of irrigation

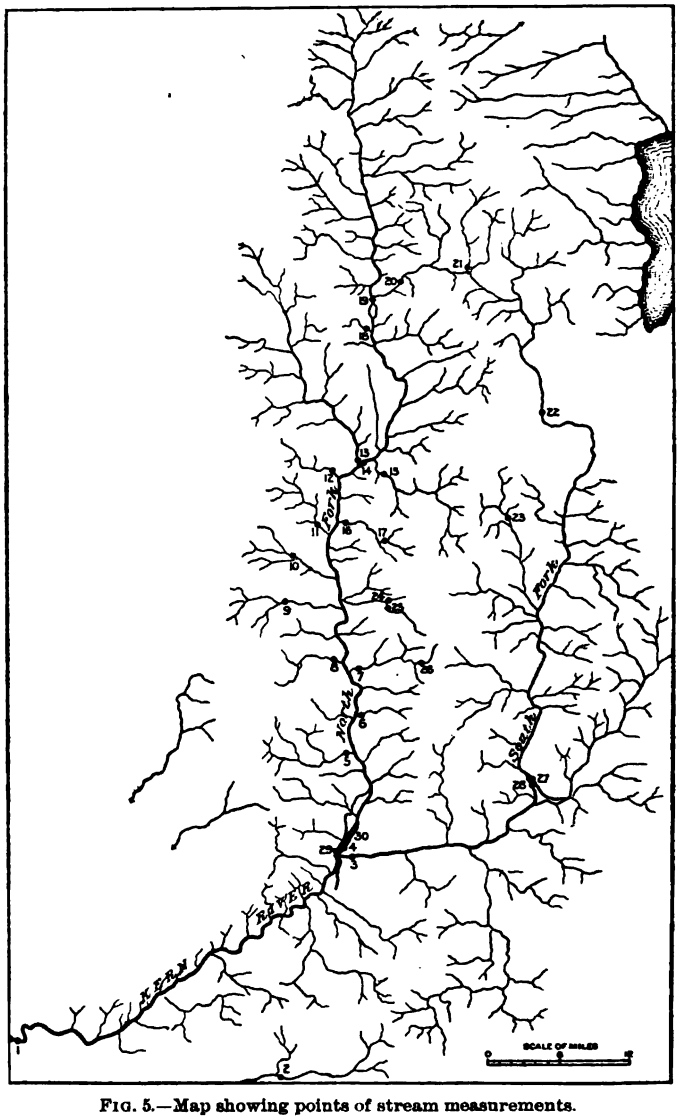


FIG. 5.—Map showing points of stream measurements.

is in many ways more convenient and perhaps not more expensive than the ordinary canal.

VOLUME.

From daily measurements continued through a series of years the run-off of Kern River Basin is well established, though the precipita-

VIEW OF GAGING STATION ON NORTH FORK OF KERN RIVER.

tion and the local discharges in the watershed are still in question. Mr. Henry Hawgood, Mem. Inst. of C. E., who has made a careful study of the physical conditions existing in the watershed of Kern River, more particularly in that of the North Fork, has estimated the annual precipitation for the North Fork above Isabella at 25.4 inches, with a run-off of 12.5 inches. This would give a mean annual flow of the North Fork of 982 second-feet. Mr. Hawgood estimates the rainfall of the entire basin at 23 inches, and the mean run-off at 7.8 inches. On this last assumption the mean discharge of Kern River at its mouth would be 1,350 second-feet. As a fact, however, for the last five years (1895–1899)—a period of exceptional drought in the basin—the stream has had a mean annual discharge at First Point of Measurement, near Bakersfield (see map, Pl. III), of 864 second-feet, or 64 per cent of the normal assumed by Mr. Hawgood.

In June, 1900, stream measurements made by the writer above Kern Lake, about 90 miles above First Point of Measurement, showed that, when considered with measurements made at the latter place and making a time allowance for the distance between, the discharge of the upper part of the stream was 99 per cent of that of the lower part. The loss in volume in the passage of 939 second-feet of water (the amount flowing in the river just above Kern Lake on June 27, 1900) from Kern Lake to Bakersfield can not be stated accurately, but in the lower reaches of the stream, from Isabella to Bakersfield, where the conditions are similar, the loss when the flow was 1,333 second-feet was determined by the writer to be 3.21 second-feet to the mile. On this basis, on June 27, 1900, the flow of the North Fork just above Kern Lake was equal to 83 per cent of the flow of Kern River at First Point of Measurement. Fig. 5 is a map showing the points where stream measurements were made, and the following table gives the dates of the measurements and the discharge in second-feet, the numbers in the table corresponding to the numbers on the map.

Discharge measurements of Kern River and tributaries at points shown on map (fig. 5).

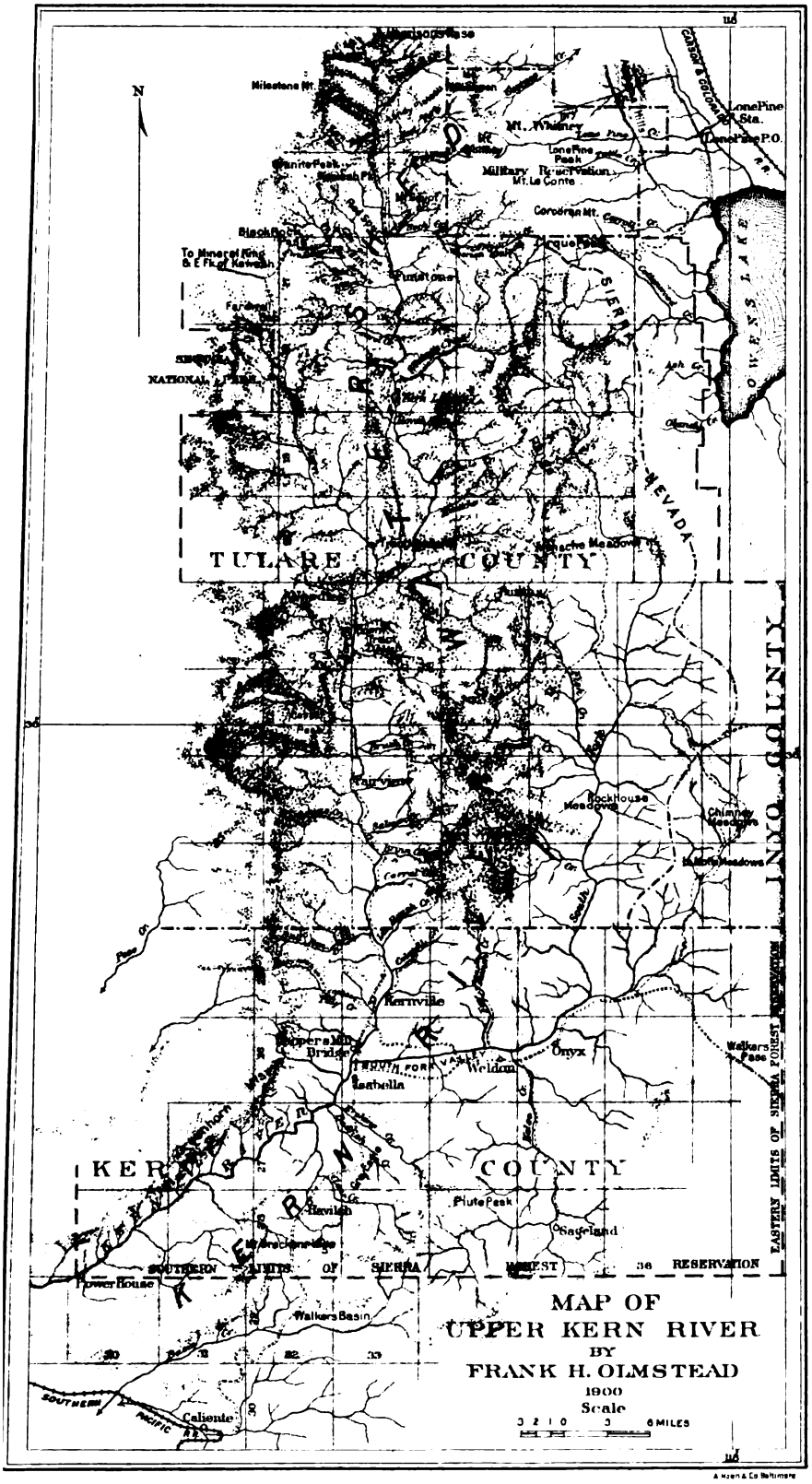
Location.	Date.	Discharge.	Location.	Date.	Discharge.
	1900.	*Sec.-ft.*		1900.	*Sec.-ft.*
1	(First Point of Measurement.)		16	June 25	2.87
			17	June 29	1.80
2	June 19	1.31	18	June 25	4.87
3	June 20	14.18	19	June 27	939.60
4	do	1,333.17	20	do	39.11
5	June 21	2.30	21	do	4.72
6	June 22	0.32	22	June 28	3.67
7	do	3.45	23	June 29	Dry.
8	do	2.92	24	do	1.04
9	June 23	5.07	25	do	8.22
10	do	5.74	26	June 30	4.05
11	do	5.19	27	July 2	2.38
12	June 24	4.26	28	do	11.05
13	do	81.00	29	July 3	1.96
14	June 25	1,154.90	30	do	7.31
15	do	8.45			

Since September, 1893, the Kern County Land Company has made careful daily measurements of the flow of Kern River at First Point of Measurement. At Rio Bravo ranch, in sec. 11, T. 29 S., R. 29 E., M. D. M., 8¼ miles above First Point of Measurement, the flow of the river has been measured for many years by the State engineer of California. It is estimated by Mr. James, chief engineer of the Kern County Land Company, that there is a loss of 50 second-feet between these gaging stations. The accompanying tables give the monthly discharge of the river at Rio Bravo ranch for the years 1878–1884, inclusive, and at First Point of Measurement for the years 1893–1900, inclusive. On page 22 is a comparative table of the estimated daily discharge of the river during 1900 at First Point of Measurement and just below Tobias Creek, which is 17 miles above the mouth of the South Fork of Kern River and about 2 miles above the mouth of Salmon Creek, and enters the North Fork from the east.

On pages 22 and 23 is a table of miscellaneous discharge measurements of Kern River and its tributaries, and on the latter page are tables of rainfall at three places in Kern River Basin.

Estimated monthly discharge of Kern River at Rio Bravo ranch.

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.	
	Maximum.	Minimum.	Mean.		Depth in inches.	Second-feet per square mile.
1878.						
November	--------	--------	400	23,812	0.19	0.17
December	--------	--------	350	21,521	.17	.15
1879.						
January	686	389	462	28,407	.23	.20
February	745	466	591	32,823	.26	.25
March	659	510	552	33,941	.28	.24
April	1,054	661	764	45,461	.37	.33
May	1,231	680	927	56,999	.46	.40
June	1,190	812	971	57,778	.47	.42
July	865	386	535	32,895	.26	.23
August	337	168	266	16,356	.13	.11
September	174	146	171	10,176	.08	.07
October	210	145	182	11,191	.09	.08
Season	1,231	145	514	371,350	2.99	.22
November	325	184	261	15,531	.12	.11
December	650	280	356	21,890	.17	.15
1880.						
January	410	315	354	21,767	.17	.15
February	380	315	370	21,282	.17	.16
March	385	349	389	23,919	.20	.17
April	3,320	395	1,557	92,648	.73	.66
May	3,560	1,615	2,659	163,496	1.31	1.14
June	4,070	2,740	3,317	197,376	1.58	1.42
July	3,140	1,550	2,196	135,027	1.08	.94
August	1,500	840	1,060	65,177	.52	.45
September	846	710	767	45,640	.37	.33
October	794	722	758	46,608	.37	.32
Season	4,070	184	1,169	850,361	6.79	.50



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Estimated monthly discharge of Kern River at Rio Bravo ranch—Continued.

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.	
	Maximum.	Minimum.	Mean.		Depth in inches.	Second-feet per square mile.
1880.						
November	830	695	767	45,640	0.37	0.33
December	1,480	790	1,063	65,361	.52	.45
1881.						
January	1,640	950	1,078	66,284	.52	.46
February	2,970	1,430	1,073	98,471	.79	.76
March	2,100	1,400	1,570	96,535	.77	.67
April	2,612	2,100	2,288	136,145	1.09	.98
May	2,710	2,060	2,362	145,110	1.16	1.01
June	2,390	1,475	1,890	112,462	.90	.81
July	1,520	710	1,126	69,235	.55	.48
August	1,200	420	627	38,552	.31	.27
September	420	320	361	21,481	.17	.15
October	360	310	333	20,475	.16	.14
Season	2,970	310	1,263	915,751	7.31	.54
November	360	300	337	20,053	.16	.14
December	410	320	350	21,521	.17	.15
1882.						
January	380	310	335	20,598	.16	.14
February	510	360	395	21,937	.18	.17
March	1,260	440	600	36,893	.30	.26
April	1,670	920	1,174	69,858	.56	.50
May	2,000	1,420	1,670	102,684	.82	.71
June	1,990	900	1,308	77,712	.62	.56
July	1,110	450	726	44,640	.36	.31
August			330	a20,291	.16	.14
September			330	a19,636	.16	.14
October			330	a20,291	.16	.14
Season	2,000	300	655	476,114	3.81	.28
November			280	a16,661	.13	.12
December			280	a17,217	.14	.12
1883.						
January			280	a17,217	.14	.12
February			350	a19,438	.16	.15
March			700	a43,041	.35	.30
April			1,170	a69,620	.56	.50
May			1,410	a86,698	.69	.60
June			1,170	a69,620	.56	.50
July			940	a57,798	.46	.40
August			470	a28,899	.23	.20
September			350	a20,826	.17	.15
October			280	a17,217	.14	.12
Season			638	464,252	3.73	.27
November			200	a11,901	.10	.09
December			200	a12,298	.10	.09
1884.						
January			350	a21,521	.17	.15
February			470	a27,084	.22	.20
March			940	a57,798	.46	.40
April			1,980	a117,818	.89	.80
May			5,860	a360,317	2.90	2.50
June			9,380	a558,148	4.46	4.00
July			5,860	a360,317	2.90	2.50
August			2,350	a144,496	1.15	1.00
September			940	a55,934	.45	.40
October			470	a28,899	.23	.20
Season			2,422	a1,756,481	14.03	1.03

a Estimated from fragmentary records by State engineer.

Estimated monthly discharge of Kern River at First Point of Measurement.

[Drainage area used in previous reports, 2,345 square miles.]

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.	
	Maximum.	Minimum.	Mean.		Depth in inches.	Second-feet per square mile.
1893.						
October	554	517	534	32,861	0.26	0.23
November	559	467	518	30,827	.24	.22
December	590	430	516	31,757	.25	.22
1894.						
January	741	562	661	40,644	.32	.28
February	1,114	604	717	39,817	.32	.30
March	1,443	762	1,001	61,541	.65	.43
April	1,892	1,209	1,495	88,952	.71	.64
May	2,208	1,228	1,607	98,798	.79	.69
June	1,719	871	1,085	64,557	.51	.46
July	1,051	400	700	43,086	.34	.30
August	549	256	335	20,565	.16	.14
September	382	172	248	14,756	.12	.11
October	363	224	279	17,178	.14	.12
November	268	230	244	14,500	.11	.10
December	805	234	470	28,908	.23	.20
The year	2,208	172	737	533,252	4.40	.31
1895.						
January	1,616	473	809	49,762	.40	.34
February	4,762	675	1,252	69,536	.55	.53
March	3,004	987	1,374	84,437	.67	.59
April	3,897	1,911	2,724	162,076	1.29	1.16
May	5,384	3,100	4,369	268,608	2.14	1.86
June	3,721	2,174	2,906	172,919	1.37	1.24
July	2,063	867	1,482	91,113	.73	.63
August	1,073	354	629	38,665	1.31	.27
September	676	290	344	20,469	.17	.15
October	612	276	327	20,106	.16	.14
November	436	308	346	20,588	.17	.15
December	447	368	408	24,779	.20	.17
The year	5,384	276	1,413	1,023,058	8.16	.60
1896.						
January	3,101	377	747	45,931	.37	.32
February	798	559	617	35,489	.28	.26
March	2,089	652	951	58,475	.47	.41
April	1,263	766	972	57,838	.46	.41
May	3,370	934	1,401	86,144	.69	.60
June	3,611	1,244	2,456	146,142	1.17	1.05
July	2,210	741	1,316	82,762	.66	.57
August	741	353	486	29,883	.24	.21
September	473	234	304	18,089	.14	.13
October	425	223	267	16,417	.13	.11
November	416	288	355	21,124	.17	.15
December	426	313	347	21,336	.17	.15
The year	3,611	223	854	619,630	4.95	.36
1897.						
January	832	305	373	22,985	.18	.16
February	2,306	516	809	44,980	.36	.35
March	2,044	688	923	56,753	.45	.39
April	4,410	1,094	2,914	173,395	1.38	1.24
May	5,342	4,054	4,580	281,613	2.25	1.95
June	4,352	1,289	2,309	137,395	1.09	.98
July	1,536	644	1,006	61,857	.49	.43
August	671	338	469	28,838	.23	.20
September	363	260	295	17,554	.14	.13
October	441	278	340	20,906	.17	.15
November	477	289	355	21,124	.17	.15
December	1,023	327	422	25,948	.21	.18
The year	5,342	260	1,234	893,248	7.12	.53

Estimated monthly discharge of Kern River at First Point of Measurement—Cont'd.

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.	
	Maximum.	Minimum.	Mean.		Depth in inches.	Second-feet per square mile.
1898.						
January	400	311	363	22,320	0.17	0.15
February	923	316	434	24,108	.20	.19
March	485	304	388	23,857	.20	.17
April	1,342	371	710	42,247	.33	.30
May	980	560	735	45,198	.36	.31
June	686	394	551	32,786	.26	.23
July	416	127	244	15,008	.12	.10
August	142	86	120	7,378	.06	.05
September	294	80	116	6,902	.06	.05
October	232	127	160	9,838	.08	.07
November	188	136	166	9,877	.08	.07
December	314	147	199	12,236	.09	.08
The year	1,342	80	348	251,743	2.01	.15
1899.						
January	361	182	263	16,172	.13	.11
February	365	258	302	16,772	.14	.13
March	4,932	247	590	36,278	.29	.25
April	1,167	593	893	53,138	.43	.38
May	1,302	576	835	51,342	.41	.36
June	2,230	809	1,331	79,200	.63	.57
July	394	229	489	30,057	.24	.21
August	240	99	156	9,592	.08	.07
September	117	89	105	6,257	.05	.04
October	229	86	160	9,838	.08	.07
November	385	183	221	13,151	.10	.09
December	781	182	277	17,032	1.36	1.18
The year	4,932	86	468	338,829	3.94	.29
1900.						
January	1,048	266	362	22,259	.17	.15
February	329	238	280	15,550	.12	.12
March	502	307	413	25,394	.21	.18
April	592	387	472	28,086	.22	.20
May	1,969	449	1,111	68,312	.54	.47
June	1,878	841	1,283	76,344	.61	.55
July	850	202	392	24,108	.20	.17
August	217	101	144	8,854	.07	.06
September	270	106	166	9,878	.08	.07
October	186	137	160	9,838	.08	.07
November	1,339	161	349	20,767	.17	.15
December	445	288	373	22,934	.18	.16
The year	1,969	101	459	332,319	2.65	.20

Thirteen-year mean of the discharge of Kern River, as compiled from the foregoing records of the State engineering department and the United States Geological Survey.

Season and year.	Discharge.	
	Sec.-ft.	*Acre-feet.*
1878–1879	514	371,350
1879–1880	1,169	850,361
1880–1881	1,263	915,751
1881–1882	655	476,114
1882–1883	638	464,252
1883–1884	2,422	1,756,481
1894	737	533,252
1895	1,413	1,023,058
1896	854	619,630
1897	1,234	893,248
1898	348	251,743
1899	468	338,829
1900	459	332,319
Thirteen-year mean	936	678,953

Estimated daily discharge of Kern River for 1900.

Day.	July.		August.		September.		October.	
	First Point of Measurement.	Just below Tobias Creek.	First Point of Measurement.	Just below Tobias Creek.	First Point of Measurement.	Just below Tobias Creek.	First Point of Measurement.	Just below Tobias Creek.
	Sec.-ft.	*Sec.-ft.*	*Sec.-ft.*	*Sec.-ft.*	*Sec.-ft.*	*Sec.-ft.*	*Sec.-ft.*	*Sec.-ft.*
1			209.08	334.00	108.31	212.80	144.95	204.32
2			190.64	338.00	115.71	310.40	142.44	205.20
3			188.25	306.20	131.89	308.00	141.92	208.16
4			193.78	252.90	147.24	274.28	147.92	204.12
5			195.49	244.38	190.01	256.00	146.64	202.80
6			188.78	234.82	159.17	253.20	154.68	202.80
7			178.86	230.62	156.86	246.40	164.04	202.40
8			169.11	230.00	241.22	244.00	164.77	200.80
9			178.32	228.66	221.09	244.00	157.39	201.20
10			171.89	228.66	212.66	234.82	153.45	202.14
11			159.38	226.14	188.13	232.06	153.30	201.80
12			161.24	226.50	175.33	232.58	151.08	201.66
13			157.67	226.00	174.77	230.44	152.02	202.00
14			149.18	224.34	170.70	222.60	156.04	201.20
15			140.79	222.34	170.66	220.30	158.50	201.20
16			130.41	222.34	165.97	220.00	156.19	200.40
17			122.57	213.30	157.06	213.76		
18			116.84	205.80	155.60	214.90		
19			116.75	201.50	151.98	211.14		
20			120.21	198.44	146.16	210.84		
21	274.25	352.58	120.16	195.36	138.63	211.00		
22	269.00	344.80	120.38	194.30	139.29	206.58		
23	274.43	340.30	121.81	192.90	142.05	206.60		
24	282.59	336.20	114.32	195.64	144.14	207.32		
25	250.55	327.72	110.85	195.00	139.79	208.90		
26	229.73	318.50	109.43	198.40	141.31	208.40		
27	224.64	308.18	110.70	194.08	153.44	207.54		
28	216.80	298.20	107.63	186.00	155.78	205.86		
29	212.69	291.34	102.99	187.20	151.31	205.32		
30	219.59	285.40	106.31	181.00	147.22	207.10		
31	222.49	282.06	103.18	178.40				
Mean			144.10	222.00	166.45	228.91		

Miscellaneous discharge measurements of Kern River and its tributaries.

Date.	Stream.	Locality.	Hydrographer.	Elevation.	Discharge.
1898.				*Feet.*	*Sec.-ft.*
July 10	North Fork of Kern River.	"A" channel, above junction with South Fork.	F. H. Olmsted		17.00
Do	do	"B" channel	do		199.00
Do	do	"C" channel	do		107.00
Do	do	"D" channel	do		8.00
Do	South Fork of Kern River.	700 feet above junction with North Fork.	do		13.00
July	North Fork of Kern River.	At mouth	do		330.00
Do	South Fork of Kern River.	At engineers' old camp.	do		18.00
Do	do	Sec. 6, T. 22 S., R. 36 E.	do		10.00
Do	do	Menache Meadows	do		5.30
Aug. 29	Kern River	First Point of Measurement.	do		115.00
1899.					
Sept. 2	do	do	do		99.22
1900.					
June 19	Basin Creek	Rankin's ranch, Walkers Basin.	do		1.32
June 20	South Fork of Kern River.	700 feet above junction.	do		14.18
Do	North Fork of Kern River.	Hooper's mill bridge.	do		1,333.17
June 21	Bull Run Creek	Near mouth	do		2.30
June 22	Tobias Creek	do	do		2.92
Do	Ant Creek	At mouth	do	3,200	.18
Do	Salmon Creek	do	do		3.45
Do	Corral Creek	Near mouth	do		.32
June 23	South Needles Creek	At Needles Peak	do	4,550	5.82

Miscellaneous discharge measurements of Kern River and its tributaries—Cont'd.

Date.	Stream.	Locality.	Hydrographer.	Elevation.	Discharge.
1900.				*Feet.*	*Sec.-ft.*
June 23	Clark Creek	Dry Meadows	F. H. Olmsted		5.19
Do	Jackson Creek	do	do		5.74
Do	Wade Creek	do	do		5.07
June 24	North Needles Creek	At Needles Peak	do		4.26
Do	Little Kern River	At junction with Kern River.	do		81.00
June 25	Tibbetts Creek	1 mile above mouth	do		2.87
Do	Harris Creek	At mouth	do		8.45
Do	Onemile Creek	1 mile below Kern Lake.	do		4.87
Do	North Fork of Kern River.	3,000 feet above junction with Little Kern River.	do		1,154.90
Do	Whitney Creek	At tunnel in divide	do		4.72
June 27	do	At lava bridge	do		39.11
Do	Creek south of Bald Mountain.		do	6,560	17.64
Do	North Fork of Kern River.	800 feet above Kern Lake.	do		939.60
June 28	South Fork of Kern River.	Menache Meadows	do		3.67
June 29	Tibbetts Creek	At 8,300 feet elevation	do		1.80
Do	North Fork of Brush Creek.	At 5,800 feet elevation	do		1.04
Do	Brush Creek	Above North Fork	do	5,600	8.22
June 30	North Fork of Kern River.	At new gaging station 4,000 feet above junction with South Fork.	do		825.25
Do	Salmon Creek	At Horse Meadows	do	7,700	4.05
July 2	South Fork of Kern River.	T. 25 S., R. 35 E	do	2,920	11.05
Do	Powers ditch	Near head	do		2.38
July 3	Neils ditch	Isabella	do		1.96
Do	Hooper's mill ditch	At gaging station on Kern River.	do		7.31

Precipitation in Kern River Basin.

DAUNT.

[Observer, Mountain Home sawmill.]

Year.	Sept.	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.
1895-96					11.91	0.83	5.13	11.01	1.30	2.70	0.65	0.50
1896-97	0.00	0.60	0.40	1.95								

KERNVILLE.

[Observer, Stephen Barton, Isabella.]

Season.	Sept.	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Total.
1895-96					3.52	0.00	1.54	0.86	0.00	0.20	2.25	0.05	
1896-97	0.00	1.15	0.41	0.85	3.40	3.60	2.57	0.10	0.00	0.00	0.00	0.00	12.06
1897-98	0.15	0.38	0.00	1.28	0.58	0.99	0.58	T.	0.54	0.00	0.00	0.00	4.50
1898-99	0.04	0.00	0.00	0.33	1.95	0.19	1.89	0.28	0.25	0.45	0.00	0.00	5.38
1899-00	0.00	0.78	0.85	0.73	0.80	0.53	0.58	0.52	0.90	0.00	0.00	0.00	5.69
1900-01	0.79	0.10	5.09	0.00									

MOUNT BRECKENRIDGE.

[Observer, G. Otterman.]

Season.	Sept.	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.
1896-97			2.20	4.30	2.00	4.67	7.72	0.00	0.40	0.10	0.00	0.00
1897-98	0.00	0.00										
1898-99					1.67	0.94						

The rain gage at Daunt is at the Mountain Home sawmill, near the divide between Kern and Kaweah rivers, and at an elevation of 6,600 feet. Kernville is at an elevation of 2,600 feet, and Mount Breckenridge at an elevation of 6,750 feet.

From the diagram (fig. 6) showing synchronous discharge and temperature observations at First Point of Measurement for the months of April, May, and June, 1897—the particular season of the year when the factor of snow enters prominently into the regimen of the river—we gather, by connecting the peak points of flow and temperature, that ordinarily it requires about forty-seven hours for the stream waters to pass from the snow line, about 100 miles up the river, to

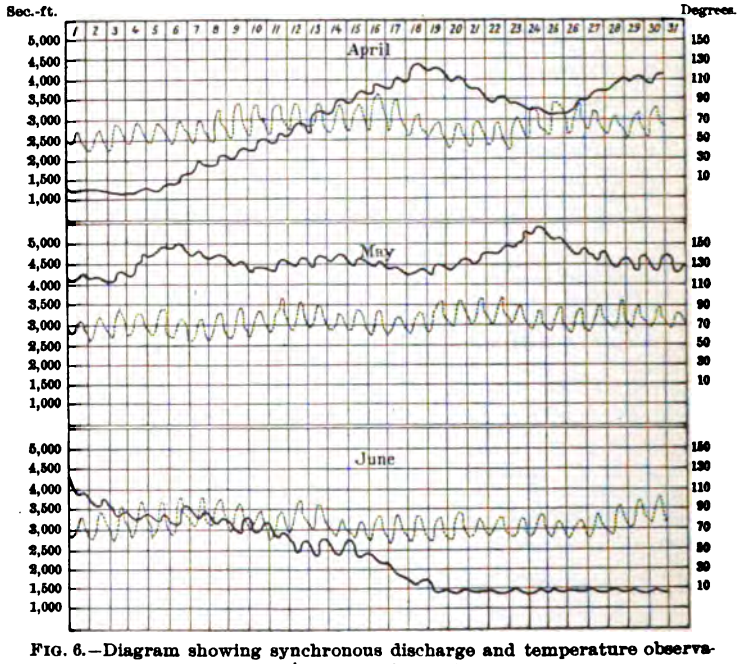

FIG. 6.—Diagram showing synchronous discharge and temperature observations at First Point of Measurement. Dotted lines are temperature curves; heavy lines are discharge curves.

First Point of Measurement. Under ordinary circumstances the melting of snow below the 7,000-foot contour would accompany a low gage, and the mean velocity would of course be relatively slow compared with a larger stream. The run-off of the heavy snows would start certainly not lower than the 8,500-foot contour, and invariably would be accompanied by a general breaking up of winter in the higher mountains, resulting in a high gage throughout the entire stream: from which we may infer that there is no marked difference in the time required for the passage of snow water between points as much as 30 miles apart to the canyon mouth. When the spring rise begins the mean velocity of the stream waters from the snow line, 60 miles

up, to First Point of Measurement must be about 1.90 feet per second. When the snow line reaches the 8,500-foot contour, 100 miles upstream, the flood velocity is approximately 3.1 feet per second. This 1.2-foot-per-second play in velocity between high and low snow water gage in the stream represents the only data available for estimating, even crudely, the velocity-slope relations during the remainder of the year.

POWER POSSIBILITIES.

Elevations and distances have been and still are to a certain extent assumptions, and were it not that the data in some form is in demand, it certainly would be desirable to possess more basic information of Kern River Basin before undertaking to even outline the power possibilities of the stream. The flood period of the year is May and June; the minimum flow occurs in September and October. During extremely dry years the flow at First Point of Measurement drops to 80 second-feet, with a probable mean for the month in which this occurs of, say, 100 second-feet.

In the reach between First Point of Measurement and Isabella the stream has been affected by losses from evaporation and seepage and by increment in flow due to the South Fork, Clear Creek, and other small tributaries. The net result of these plus and minus factors is estimated to be a loss between these points of 122 second-feet. Calling this loss 120 second-feet, and taking the minimum flow at First Point of Measurement (80 second-feet), we have, as the least flow of the river at the latter place, 200 second-feet. Between Isabella and First Point of Measurement the fall is about 1,900 feet, giving more than 42,000 theoretical horsepower. At Isabella the mean flow for the full year 1899 was 588 second-feet, for 1898 it was 469 second-feet, and for 1897 it was 1,353 second-feet. Below Isabella the topography on the right bank is not unfavorable for the construction of a large canal, but the immense bowlders which cover the southern and western sides of the Greenhorn Mountains, and which are constantly in transit down the slopes, are not an economical factor in canal construction.

Of the total flow of Kern River at First Point of Measurement 80 per cent passes Funstons on the North Fork, or a point of equal elevation (7,050 feet) on the South Fork, which would be about midway between the lower end of Menache Meadows and the mouth of Fish Creek. The theoretical power possibilities of the stream below these places would be, then, about 500,000 horsepower, which, for a watershed of 2,349.3 square miles in an arid region, evidences the remarkable advantages of the stream, so far as grades and discharge are concerned, for electrical-power development.

The Kern River Company, which has been engaged for the last four years in securing rights of way (now approved by the Department of the Interior), and in arranging preliminaries incident to the construc-

tion of so large a plant, has as yet accomplished nothing in the way of actual building. A plan of the reservoir site of the company is shown in fig. 7. The diversion works of their power canal will be in

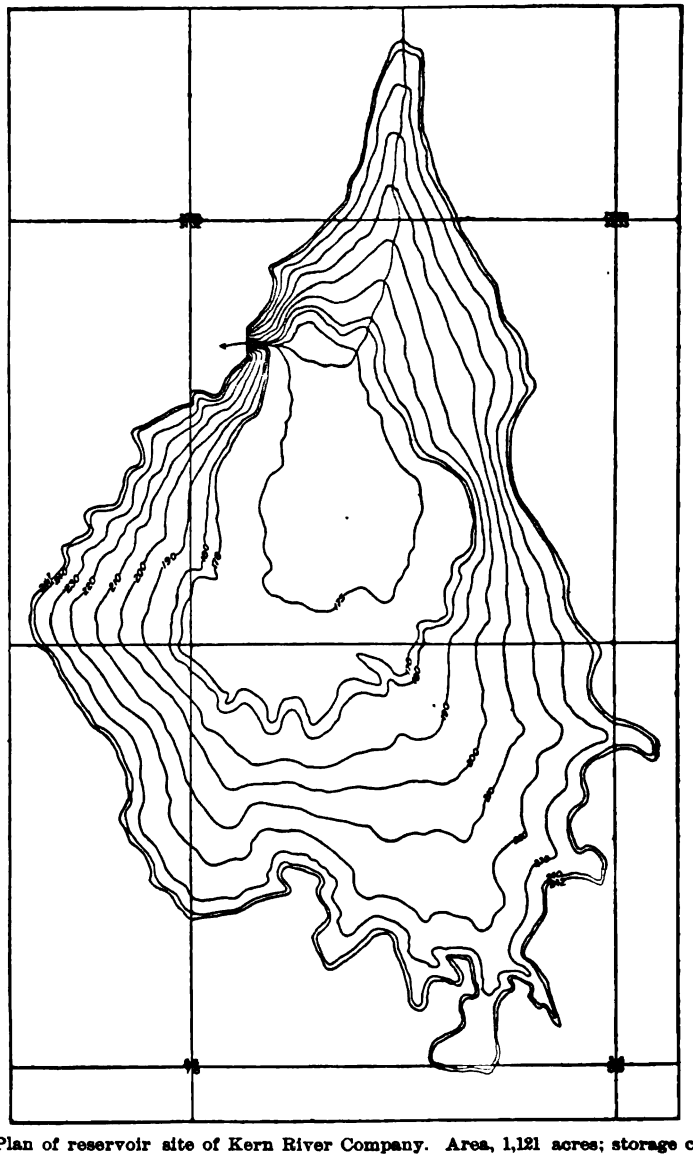

FIG. 7.—Plan of reservoir site of Kern River Company. Area, 1,121 acres; storage capacity, 42,000 acre-feet; height of dam, 72 feet.

the lower end of the town of Kernville, where the river forks. The West Fork is to be deepened so that during low stages this branch will gather all of the water of the river, and a low training wall, over which floods of the river will pass, will be built. The head gate will

be 3,600 feet distant, on bed rock, and of solid masonry. The standard cross section of the canal for the first 10 miles is 25 feet wide at bottom, 7.5 feet depth of water, and on a grade of 1.058 feet to the mile. At a distance of 4½ miles from the head gate the only considerable structure on the line is encountered—the bridge over Kern River. After passing that point the cross section of the canal is reduced and the grades are increased accordingly.

The plans of the company are based upon a minimum flow of Kern River past their diversion works of 400 second-feet. This, considering the storage proposed on Salmon Creek, which is described further on, is a conservative estimate for any ordinary series of years, but could not have been maintained in 1899–1900, although in nine out of ten years it easily could be increased to 500 second-feet. The plans are based on an available head at the wheels of 230 feet during extreme high water, with an additional head of 40 feet when the river is at its minimum stage. Mr. H. Hawgood, the chief engineer of the Kern River Company, has designed, as a generating unit, a water wheel of 1,500 horsepower, with an alternator of 1,030 kilowatts capacity. There are to be nine of these units—eight for daily service and one to be held in reserve. For the additional 40-foot head possible during low water, there will be a 750-horsepower wheel and one 550-kilowatt alternator. On this basis, at no time would there be supplied to the wheels less than 11,200 horsepower. The impulse type of wheel would give an efficiency of 80 per cent, the alternators an efficiency of 95 per cent, and the step-up transformers an efficiency of 98 per cent, thus delivering to the line 8,342 horsepower. Assuming 5 per cent average loss in transmission (a larger allowance than the 80-mile carriage of the Southern California Power Company's current indicates is necessary), there would be delivered at the Los Angeles power house 7,825 horsepower, and with 98 per cent efficiency in the step-down transformers there would be 5,720 kilowatts for daily distribution in the city. This delivery could be increased 50 per cent for an hour to take care of the maximum load, which occurs between the hours of 5 and 6 p. m. Speaking conservatively, and having in mind the ruling rates elsewhere on the Coast, this 7,668 horsepower should command 1⅓ cents per horsepower per hour. During the irrigating season of 1900 the municipality of Los Angeles paid 2.2 cents per horsepower per hour for power registered between the hours of 11 p. m. and 5 p. m., and 4.4 cents per horsepower per hour for power registered between the hours of 5 p. m. and 11 p. m. In large plants working eighteen hours a day, and with good bituminous coal at $4 a ton, the cost of 1 horsepower per year would be $85, and in small plants, where the attendance factor is relatively greater, the cost would be $150. At the rate of 1⅓ cents the cost of a horsepower per year of three hundred and thirteen working days is $75. At this rate the gross annual revenue of the plant would be $768,000.

The company's estimate of the cost of construction of the plant and lines for the transmission of the current to Los Angeles is, approximately, $1,333,000. Using direct-connected motor and centrifugal pumps, with 30 feet depth of wheel pit and 20 feet additional lift, and assuming the duty of water to be 8 acres per miners' inch, 1.42 horsepower would irrigate 100 acres at the rate of, say, $2.50 per acre per annum. This would be on the basis of a 50-foot lift and a rate of 2 cents per horsepower per hour for the current. The efficiency of pumping plants ranges from 20 to 68 per cent—frequently plants are condemned solely because of a lack of skillful assembling of the units.

Electric motive power possesses many advantages over either steam or gasoline plants for pumping water, and in actual cost of service it may be questioned whether it is not to be compared with many well-managed canal systems of the State. The Kern Land Company has twenty-five pumping plants in operation near Bakersfield, each delivering from 1,400 to 2,000 gallons a minute. These plants each consist of four wells in a line, sunk to a depth of 80 to 130 feet through strata of alluvial loam, clay, and water-bearing sand. The wells are cased with galvanized iron, 13 inches in diameter, No. 16 gage, perforated with vertical slits opened one-sixteenth of an inch. It was the practice of Mr. Lewis A. Hicks, the engineer in charge of this work, to land the casing in clay, and to perforate for all sand below a depth of 30 feet. Surface water was generally encountered at about 15 feet. In each plant the four wells are 6 feet apart on centers, and by experiment it was found that the flow from the four was generally a little more than double that from one. The total lift for most of these plants was about 35 feet.

After many experiments to determine the pump best suited to these conditions, a centrifugal pump, connected directly to the motor and working on a vertical rod, all thoroughly bolted to steel framework, was found to give the greatest efficiency and the least trouble in operation—one attendant looks after ten plants, and there have been months when the plants were in operation 98 per cent of the time. The farmers under the Kern Land Company's water supply are taxed 75 cents per acre-foot for irrigating water from these electrically driven pumping plants, and as the land requires 2 acre-feet annual irrigation the expense is $1.50 per acre per year. It is not probable that the rate of 1½ cents per horsepower per hour could be maintained in Antelope Valley or farther south in San Fernando Valley for irregular demand, but there is no good reason why a 2-cent rate could not be maintained by any of the Kern River companies.

The 68 per cent efficiency obtained in many of the Kern Land Company's plants deserves more than passing notice. In electrically driven pumping plants 55 per cent efficiency (reckoned from the meter consumption to the foot-pounds raised) is very good, but when this is raised to 68 per cent, the method employed to obtain this result

merits a detailed description. The following is quoted from one of Mr. Hicks's reports:

The pumps are of the Pit type, provided with 10-inch outlet, automatic balance, heavy shaft, and runners of special curvature adapted to the speed and height of lift, and a delivery of 5 to 6 cubic feet per second. The bearings are provided with sight-feed oil cups, and the feed on the upper bearing is upward against the water with which the stuffing-gland chamber is filled. The chamber is provided with a gage glass to enable the attendant to note any leak through the packing. The pump is bolted to cast-iron pedestal set on a wooden base, and the steel angles which support the motor are attached to the same pedestal. The assembling of the pumps and frame was accomplished in the shops, and the completed unit was hauled to its destination and lowered into place on the floor of the pit, only requiring to be guyed to a perpendicular position in the anchor frame to be ready to receive the motor. The adjustment of the thrust of the pump can be altered so as to carry the entire weight of the motor and shafting or such portion of it as may be desirable. It has been found preferable to separate them with 100 to 200 pounds down thrust, as there is a tendency to bow out the shaft between bearings if it is thrown into compression with resulting vibration. The motor is provided with adjusting screws, so that its position in vertical or horizontal planes can be easily changed with a hand wrench. The pump, motor, and frame constitute a self-contained unit, so that any settlement of foundation does not

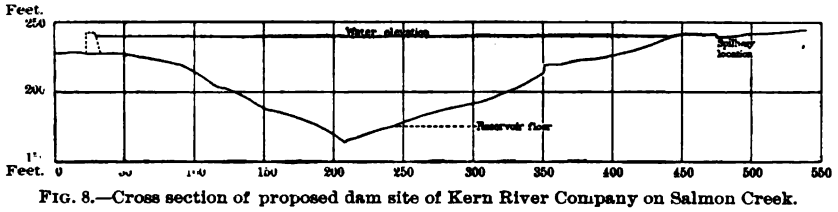

FIG. 8.—Cross section of proposed dam site of Kern River Company on Salmon Creek.

alter the relative position of either machine, and can be quickly rectified, should it occur, by guying the frame back to a vertical position. The motors used for this installation are the ordinary type of 30-horsepower inductive motor, equipped with special end shields to adapt them for vertical use. The oiling is accomplished by means of centrifugal force, which is utilized to lift the oil from the inside periphery of a revolving cup to the top of the bearings, whence it returns to the oil cups through oil grooves along the shaft. The motors are wound for a potential of 550 volts, and as the transformers at these points can be connected up to 605 volts, the effective heating overload is greatly reduced.

The Kern River Company's transmission line from the power station at the mouth of Clear Creek to Los Angeles would be 105 miles long, map measurement, and 108 miles when the vertical departures are considered. The line begins at an elevation of 2,450 feet, and its southern terminal at Los Angeles is 350 feet above the sea. A number of mountain ranges would be crossed, but a large proportion of the line would be over good ground, and it is believed that the highest elevation reached (6,500 feet, at Tehachapi) will offer no serious obstacles to a daily inspection of the pole line.

A reservoir on Salmon Creek, a tributary of Kern River, is planned by the Kern River Company for the storage of 47,000 acre-feet of

water. About 20,000 acres can be made tributary to this intake, and it is hoped that with this catchment basin and the precipitation from an elevation of 7,700 feet on this divide the reservoir will supplement the flow of Kern River to 400 second-feet during the particular season of any year when the normal flow is below that figure. Fig. 8 is a cross section of the dam site on Salmon Creek. It is in a

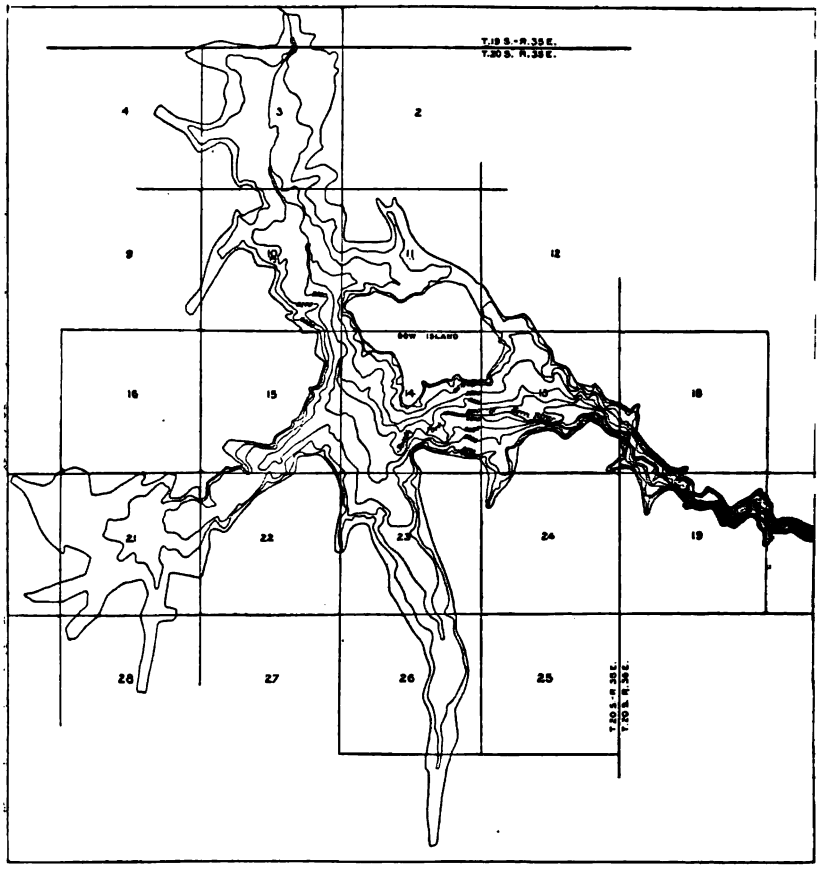

FIG. 9.—Plan of reservoir site of Kern-Rand Electric Power Company. Area, 3,562 acres; storage capacity, 63,722 acre-feet; height of dam, 65 feet above stream bed.

granite canyon, with clean bed rock on bottom and sides. The width at the bottom is 125 feet between walls; the top width at the 75-foot level would be 390 feet. A rock-fill dam is estimated to require 26,000 cubic yards of material and to cost $80,000.[1]

The utilization of the higher reaches of Kern River for power purposes will certainly be accomplished in time, but for the present,

[1] Reservoirs for Irrigation, Water Power, and Domestic Water Supply, by J. D. Schuyler.

and with the disinclination of electrical engineers to recommend the transmission of power to distances greater than 100 miles, it is doubtful whether the market outside of Los Angeles will warrant the outlay; and the latter city is so far away from the upper river as

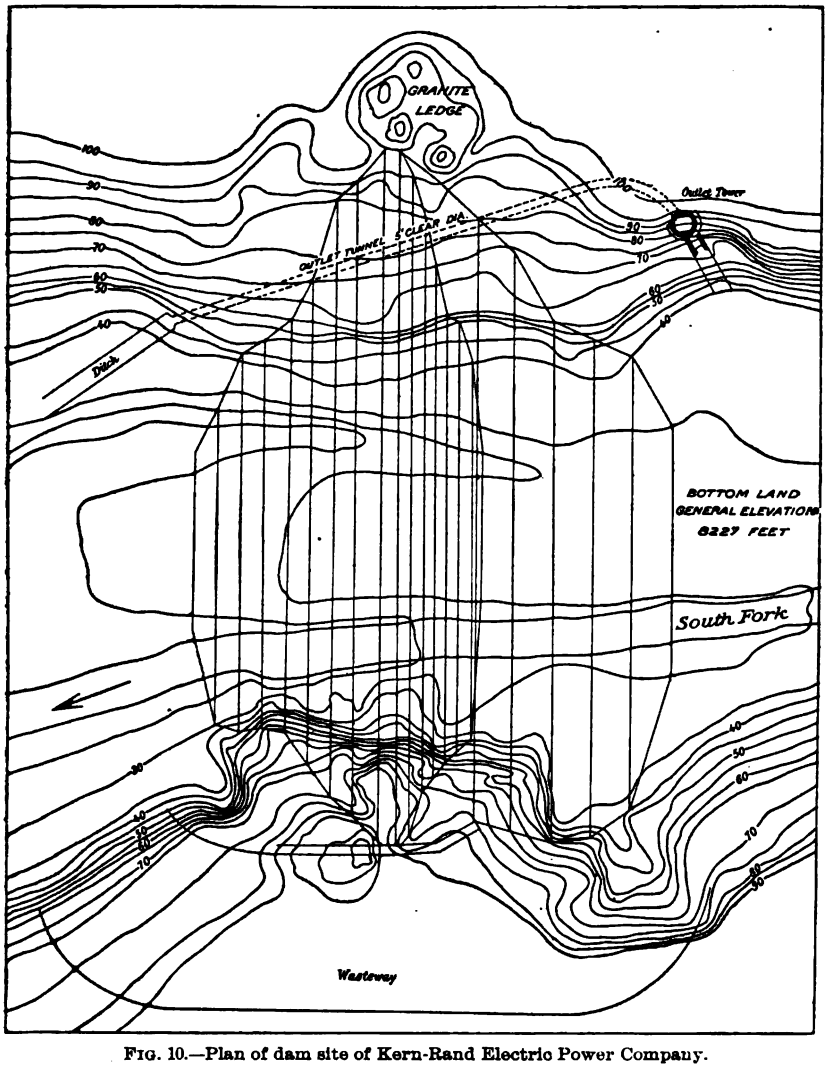

FIG. 10.—Plan of dam site of Kern-Rand Electric Power Company.

to preclude, for the present at least, supplying it with power from that source.

The Kern-Rand Electric Power Company, of Los Angeles, purposes to construct a rock-fill dam on the South Fork of Kern River, at Menache Meadows, and an initial power station near the upper end

of South Fork Valley, about 35 miles farther down the stream. At some recent geological time Menache Meadows must have been an immense mountain lake. A dam 65 feet high would throw the water back 6 miles from the canyon mouth, in two arms 6 miles apart. A dam of that height would store 63,700 acre-feet of water at a mean depth of 18 feet. There is no question that in an ordinary year the drainage area tributary to the South Fork and above the Meadows (165 square miles) would furnish ample storage water for this reservoir, although it is probable that in a season like that of 1899–1900 there would be a shortage. There is no reliable data for determining the minimum precipitation in Menache Meadows, for generally the snows lie so deep over the mountains surrounding the basin as to make it impenetrable during at least the colder half of the year.

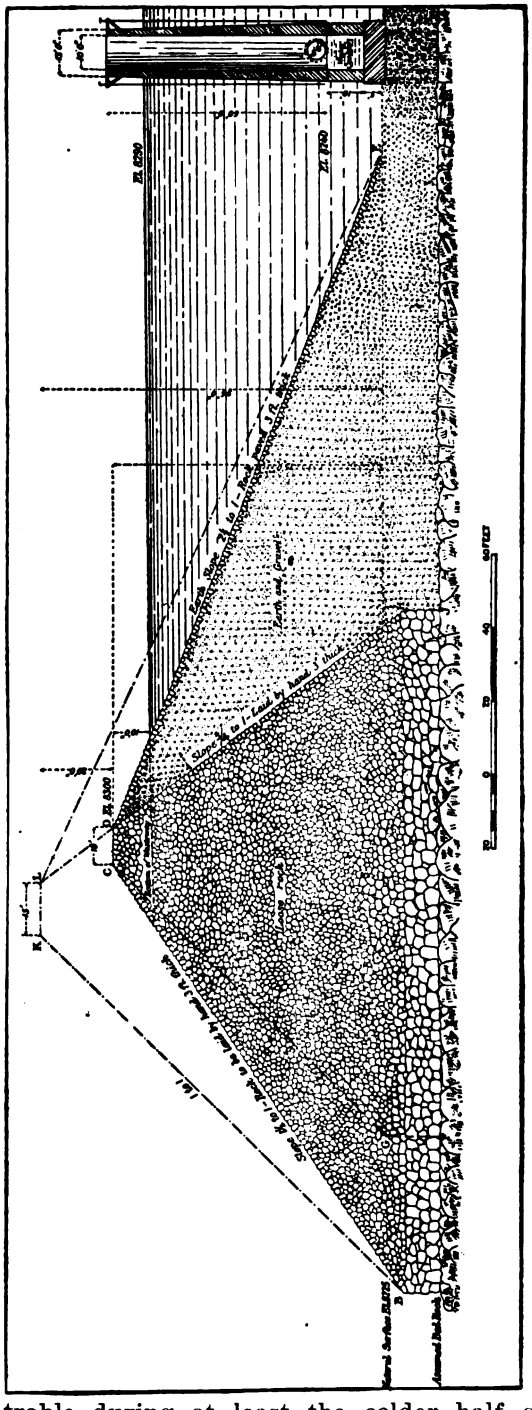

FIG. 11.—Sectional elevation of dam proposed by the Kern-Rand Electric Power Company. For dam 75 feet high above bed rock, build on lines *F*, *G*, *C*, *D*, and *E*. If it is desired to increase capacity of reservoir in future, build on lines *A*, *B*, *C*, *D*, and *E*. Dam built on lines *B*, *K*, *L*, and *E* would nearly treble the capacity of the reservoir.

Capacity of Menache Meadows reservoir site.

Height above base of dam.	Surface area.	Capacity.
Feet.	*Acres.*	*Acre-feet.*
10	22	110
20	146	954
30	812	4,563
40	1,865	18,827
50	2,599	40,732
60	3,254	79,885
70	3,814	105,236
80	4,420	146,419
100	5,830	248,852

Fig. 9 is a plan of the reservoir site of the Kern-Rand Electric Power Company in Menache Meadows, fig. 10 is a plan of the dam site, and figs. 11, 12, and 13 are details of the dam proposed. The material at the dam site is apparently hard granite, overlain with earth, sand, and gravel. The dam site is at an elevation of 8,200 feet above the sea, and consequently snow and frost prevail there during the winter months. There is no wagon road to the dam site, and the cost of making one would add very materially to the cost of the dam. The drainage area tributary to this reservoir being all above an elevation of 8,200 feet, naturally has the greater part of its precipitation in the form of snow. This makes the run-off an uncertain quantity, and leaves the proper height for the dam a difficult question to decide.

Taking all of these considerations into account, the engineers of the company were led to select a loose rock-fill dam faced with earth as the most economical and serviceable for the locality. On account of the possible action of frost, flatter slopes were given the rock faces than are absolutely necessary to make the dam heavy enough to resist the water pressure. So far as possible, the material found in the immediate vicinity of the dam will be used in its construction, and the outlet, gates, and connections have been designed so that all of their parts can easily be transported on mules over a steep mountain trail. The crest of the dam was fixed at 65 feet above the natural surface of the ground at the dam site. This height can, however, be increased to 95 feet, if it is found that a larger reservoir capacity could be supplied, at a proportionate increase in cost. It is proposed to excavate all of the rock required in the construction of the dam on the north side, forming a suitable wasteway in the solid rock, capable of discharging a stream of water 100 feet wide and 9 feet deep at a velocity of 6 feet per second—a volume of 5,400 cubic feet per second. As each foot rise of water over the bottom of the wasteway represents about 5,000 acre-feet, 9 feet rise of water in the wasteway would increase the volume in the reservoir to 45,000 acre-feet. This, in addition to the water running through the wasteway and outlet, would make more than 50,000 acre-feet, which, if it all came in twenty-four hours, would represent a run-off of about 5½ inches. It

is believed that a wasteway of this size would make the dam absolutely safe against any possibility of the water ever flowing over the top. The capacity of the reservoir with a dam 65 feet high, as now

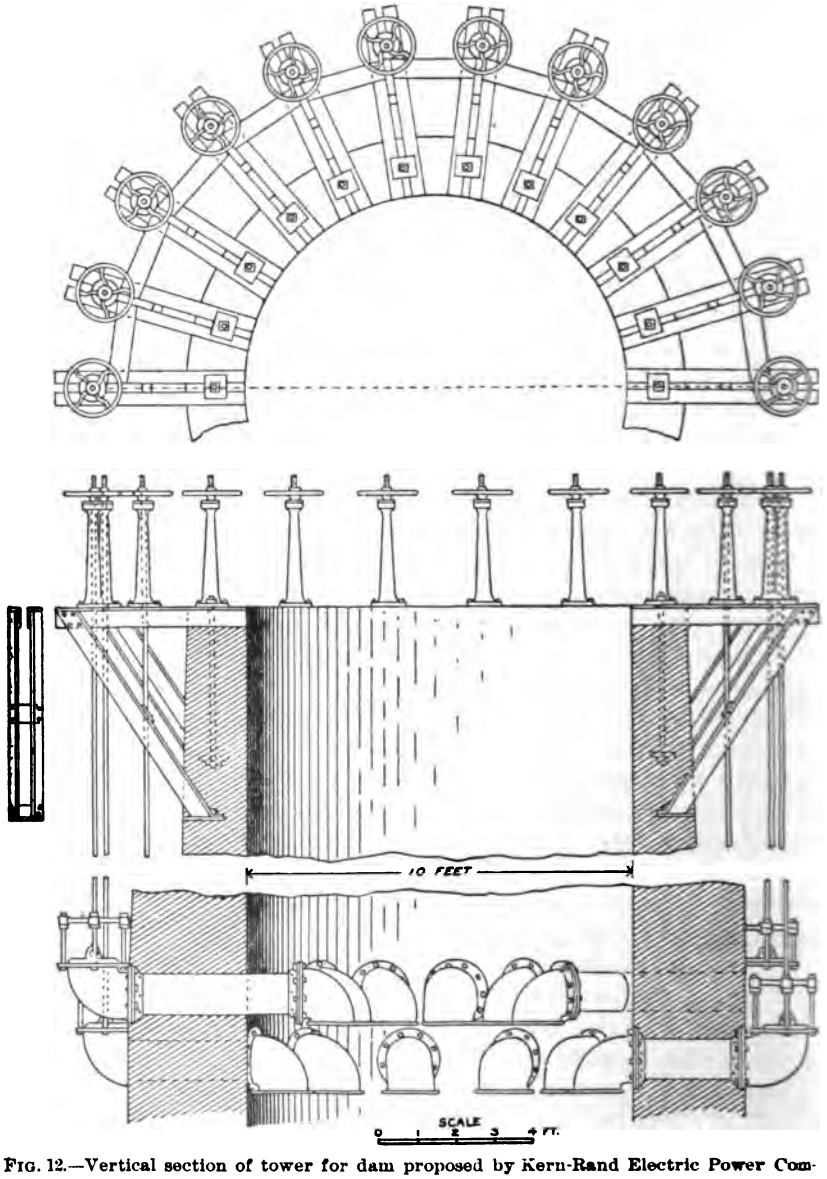

FIG. 12.—Vertical section of tower for dam proposed by Kern-Rand Electric Power Company, showing arrangement of gates.

proposed, would be 63,700 acre-feet; and if the dam were raised hereafter to a height of 85 feet the capacity of the reservoir would be nearly trebled.

It is proposed to take the water from the reservoir by means of a tunnel through the solid rock on the south side of the dam. The masonry or concrete tower (see figs. 11, 12, and 13) is designed as an inlet to this tunnel and to accommodate the placing of the gates in

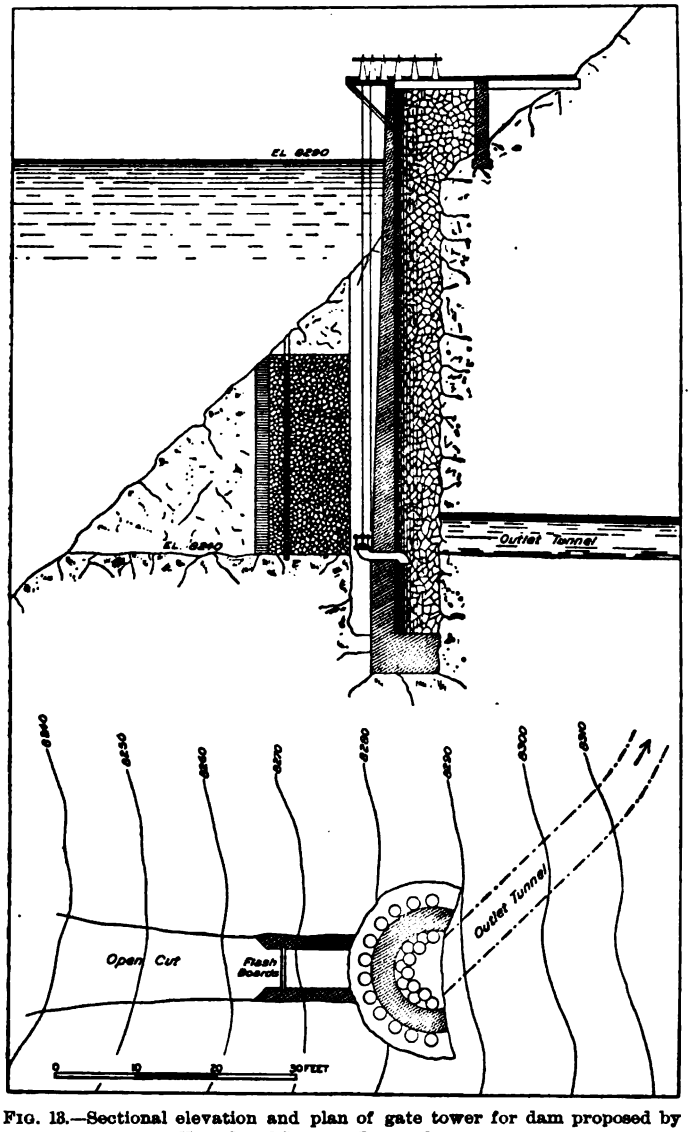

FIG. 13.—Sectional elevation and plan of gate tower for dam proposed by Kern-Rand Electric Power Company.

the most simple and economical manner. The gates are so simple and so easily opened and closed that it has not been considered necessary to provide for their repair under a full head of water in the reservoir. Fig. 12, however, shows a simple and economical way of

shutting off the water after it has fallen to an elevation of 8,260 feet, or 35 feet above the base of the dam. This would prevent the necessity of emptying the reservoir for the purpose of making repairs to the gates.

Following are the estimates for three sizes of the dam proposed:

Estimate of cost of a 75-foot dam with slopes 1 to 1 and ¾ to 1.

Loose rock, 73,000 cubic yards; or 43,800 cubic yards solid at $1.50	$65,700.00
Earth, 55,870 cubic yards at $0.25	13,967.50
Extra labor on 5,000 cubic yards laid by hand, at $1	5,000.00
Tunnel, 350 feet at $12	4,200.00
Tower, gates, and connections	15,000.00
Guide walls, etc., at wasteway	15,000.00
	118,867.50
Engineering and contingencies, 10 per cent	11,886.75
Total	130,754.25

Estimate of cost of 75-foot dam with slopes 1½ to 1 and ¾ to 1.

Loose rock, 91,000 cubic yards; or 54,600 cubic yards solid at $1.50	$81,900.00
Earth, 55,870 cubic yards at $0.25	13,967.50
Extra labor on 6,000 cubic yards laid by hand, at $1	6,000.00
Tunnel, 350 feet at $12	4,200.00
Tower, gates, and connections	15,000.00
Guide walls, etc., at wasteway	15,000.00
	136,067.50
Engineering and contingencies, 10 per cent	13,606.75
Total	149,674.25

Estimate of cost of 95-foot dam with slopes 1 to 1 and ¾ to 1.

Loose rock, 110,000 cubic yards; or 66,000 cubic yards solid at $1.50	$99,000
Earth, 70,000 cubic yards at $0.25	17,500
Extra labor on 7,000 cubic yards, at $1	7,000
Tunnel, 350 feet at $12	4,200
Tower, gates, and connections, with bridge to tower	17,000
Guide walls, etc., at wasteway	20,000
	164,700
Engineering and contingencies, 10 per cent	16,470
Total	181,170

The foregoing estimates are based on the supposition that the rock will be hard enough to stand vertically around the tower and that the tunnel will not need lining.

The transmission line from the power plant in sec. 14, T. 25 S., R.

35 E., M. D. M., to Randsburg, 43 miles distant, would be over as fine a country for a pole line as can well be found—easy slopes, virtually unimproved and without tree growth. With very few exceptions a wagon could be driven, without road work, along the transmission line from the power plant to Randsburg. The highest elevation reached on the pole line would be at Walkers Pass, 5,320 feet above sea level, and at that point the snow lasts only a few days and would never interfere with the regular patrol of the line.

The following is an approximate estimate of the cost of generating and transmitting electric current from the South Fork power house, for a delivery in Randsburg of 900 horsepower, with conduit and pole-line capacity for 450 additional horsepower:

Estimate of cost of generating and transmitting electric current from South Fork power house to Randsburg.

Diversion in bed rock and 100 feet of rock channel	$4,000
Regulating gate	1,000
Riveted-steel pipe, 8,000 feet of 30-inch, gage No. 12, at $2.70	21,600
Pipe work in canyon, 8,000 feet	10,000
Flume, 3,500 feet at $3	10,500
Tunnel, 1,300 feet at $10	13,000
Canal in earth, 3,000 feet, with concrete lining	6,750
Canal in rock, 1,594 feet at $1.50 per foot run	2,391
Siphon, 1,000 feet at $3	3,000
Penstock, 4,000 feet at $4	16,000
Water wheels	6,000
Power houses, two, fireproof	6,000
Wire for transmitting current, 900 horsepower, delivered 43 miles	13,000
Telephone	2,000
Line poles (43 miles, 40 to the mile), and placing same	20,000
Electric machinery	30,000
Distribution of current at Randsburg	5,000
	170,241
Contingencies, 15 per cent	24,536
Total	195,777

A landslide has blocked the canyon and created a lake of about 40 acres area on the North Fork of Kern River just below the mouth of Whitney Creek and above the mouth of the Little Kern. This lake, known as Kern Lake, the California State engineering department, in its investigations of the Kern River drainage basin, has considered as a possible reservoir site. At the lower end of the lake the cliffs tower almost vertically above it to heights ranging from 2,000 to 3,000 feet, and estimates have been made for the blasting of large fragments into the dam site, forming a loose rock-fill dam. The capacity of the reservoir at the 220-foot level would be 46,000 acre-feet. The State

engineering department considered the possibility of paving the mass of rock thus thrown down to a uniform surface and covering it with asphalt. The width of the canyon at the site is only 100 feet at the bottom and 400 feet at a height of 230 feet above the stream bed. There would be no question about the ability of the drainage basin to fill the reservoir annually, and if it is possible to construct such a dam at this place the site probably would be of value for replenishing the late summer flow for power and irrigation.

RECONNAISSANCE OF YUBA RIVER, CALIFORNIA.

By MARSDEN MANSON.

WATERSHED.

Yuba River is a tributary of Feather River, which it enters at Marysville, 30 miles above its mouth. It drains about 1,357 square miles of the western slope of the Sierra Nevada, comprising portions of Sierra, Nevada, Plumas, and Yuba counties. The extreme length of the watershed is about 60 miles, the extreme width 36 miles. In addition to the length given there are about 11 miles of channel in the valley between the foothills and Feather River. In size Yuba River is fourth in the Sacramento Valley. Its extreme low-water discharge is about 360 cubic feet per second,[1] its mean winter discharge 1,500 cubic feet per second, and its flood discharge 26,000 cubic feet per second.[2] For the lower 10 miles of its course in the foothills the river is greatly clogged with débris from hydraulic-mining camps (estimated at many million cubic yards), and is between levees which have been raised from year to year to meet the overflow caused by the filling up of the area between them. The channel of the river in the lower foothills has been filled with cobbles and gravel to a depth of more than 100 feet. (See Pl. IV, *B*.) From the foothills to the mouth of the river at Marysville the channel is over a surface of gravel, sand, and clay, recently built up from the mines above. The channels are irregular and change from winter to winter and sometimes during the summer. It is therefore impracticable to establish low-water gaging stations which would serve for more than one summer and fall and which would be suitable for winter or flood-stage gagings.

[1] This is not as small as the natural discharge would be. The large mining companies—the South Yuba Canal Company, the North Bloomfield Gravel and Mining Company, the Milton Excelsior Water and Mining Company, the Eureka Lake and Yuba Canal Company, and others—store large volumes of water during the winter and spring months for use during periods of low water in the late summer and the early autumn.

[2] Extreme flood discharge estimated by Mr. Hubert Vischer, Asst. Engr., U. S. Engrs., at 125,000 cubic feet per second. H. R. Doc. No. 431, Fifty-sixth Congress, second session, p. 12.

The changes in the bottom and in the position of the channel are so great that the gagings at the flood stages of the river would be unsatisfactory, and if undertaken from boats would be highly dangerous, if not impossible.

The drainage basin is subdivided into five small basins, namely, North Fork, with a drainage area of 491.6 square miles; Middle Fork, with a drainage area of 218 square miles; South Fork, with a drainage area of 360 square miles; Deer Creek, with a drainage area of 89.6 square miles; and Dry Creek, with a drainage area of 105.5 square miles. In addition to these an area of 92.5 square miles drains into the main stream above the 100-foot contour. Dry Creek joins Yuba River from the north just as it leaves the foothills. The other streams unite in the mountains. The forks are perennial in flow, but the

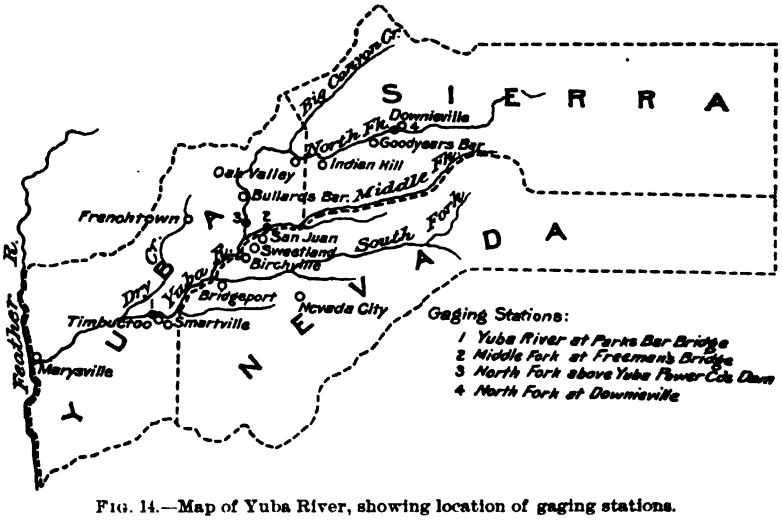

FIG. 14.—Map of Yuba River, showing location of gaging stations.

discharge of the two creeks mentioned (Deer and Dry) becomes insignificant in the late summer and early autumn.

As they merge into the valley the Sierra foothills have an elevation of about 100 feet above tide. The watershed rises gently, in rounded and broken mountains, to the crest of the Sierra, which at the headwaters of the Yuba is at a mean elevation of about 8,200 feet, with peaks rising to a height of 9,100 feet. From Mount Lincoln—a peak common to the watersheds of Yuba, American, and Truckee rivers—to a point about 2½ miles northeast of Mount Webber, the summit of the Sierra divides the watershed of Yuba River from that of Truckee River, which discharges into Humboldt Basin. Farther north from Mount Webber there is a secondary crest which divides the watersheds of Yuba and Feather rivers, the watershed of the latter stream reaching farther east, to a less elevated divide in which the passes are lower than those of the easterly crest.

A. HEAD OF SOUTH FORK OF YUBA RIVER AND LAKE SPAULDING.

B. YUBA RIVER IN THE FOOTHILLS.

TOPOGRAPHY.

The western and lower portions of the drainage area are composed of slates and kindred rock, very much eroded and merging into the gravel and alluvial deposits of the great valley of California. The upper portions of the basin are composed principally of lavas and granites, all deeply eroded, particularly the lavas. Some idea of the magnitude of the erosion may be obtained when it is considered that it has required at least 4,000 (possibly 6,000) cubic miles of denuded materials to fill the great valley of California to its present level, and that most of this has come from the Sierra Nevada.

A stratum of serpentine traverses the watershed of Yuba River in a direction generally parallel with the crest of the Sierra. It is intersected by the North Fork at Goodyears Bar, by the Middle Fork near Moores Flat, and by the South Fork just east of Washington, and leaves the drainage basin of the Yuba, passing near Towle Station on the Central Pacific Railroad. This stratum is generally softer and more easily eroded than adjoining strata, and through it the canyons of the various forks are upon lighter grades than immediately above and below, and they generally are wider. This softer material also controls the loci of longer and more deeply eroded tributaries, which afford approaches to the main canyons for roads and trails. This stratum is of further interest because it is the dividing line between the auriferous strata in the watershed. To the west of it the mines are more extensive, the occurrences of gold-bearing rock to the east being irregular and difficult to trace.

The middle and upper portions of the watersheds of the three forks differ materially. The North Fork rises in lavas which vary much in composition and hardness, but which generally afford a deep soil for timber and shrub growth. The Middle Fork rises in similar lavas and in granite. The mean elevation of the crest of the Sierra at the head of these forks is about 8,200 feet. The main and tributary streams fall rapidly, and their canyons head well up in the mountains. The sides of these canyons are covered with timber and brush, which, with the deep soil, retain the moisture and feed numerous perennial springs. (See Pl. V, *A*.) In the case of the North Fork this is particularly noticeable. The forests of its watershed make it a reliable and constant stream. The mean annual precipitation upon the watersheds of the North and Middle forks is about 54 inches. Warm rains on soft snow sometimes give a high flood run-off, but snow remains on the higher peaks until midsummer. Reservoir sites are not numerous; they will be mentioned later. The headwaters of the South Fork lie upon a broad granite surface, into which the streams have not cut deeply until the main stream reaches a point 16 miles from the summit, where it drops rapidly into a deeply eroded canyon. The eastern or upper edge of the drainage area has a mean elevation about the same as the other forks, but the 5,000-foot contour is about

20 miles to the westward. This broad surface has been denuded by glacial action, and the harder nature of the granite has not permitted a deep soil to form. The area is therefore less heavily timbered than the drainage areas of the other two forks, and its accessibility has caused it to suffer more severely from the ax of the lumberman. This topography gives a broader and more gently sloping surface than characterizes the headwaters of other Sierra Nevada streams. The surface is marked by nearly 100 glacial lakelets and valleys, affording many excellent reservoir sites which have been or are being utilized. This elevated watershed receives a mean annual precipitation of 60 inches, most of which is in the form of snow. The slow melting of the snow maintains the discharge of tributaries until June or July, which, with the natural and artificial reservoirs, makes the South Fork of the Yuba a highly valuable and reliable source of water supply.

NATURAL STORAGE OF WATER IN YUBA RIVER BASIN.

Precipitation upon the drainage basin of Yuba River is dependent upon the southerly or winter extension of the north temperate rain belt. During the summer months the more northerly position of this belt leaves California in the comparatively rainless region between the north temperate and equatorial rain belts. The rains and snows, therefore, fall from October to April, with little or no rainfall of moment from May to September, so that during the latter months the streams depend upon either natural or artificial storage. Natural storage is by snow and the slow run-off of water retained in afforested and brush-covered soils. On the South Fork artificial storage has reached a very effective stage. The precipitation ranges from 20 inches at Marysville, in the valley, to 70 inches at the summit of the Sierra.[1]

Snow storage of water is depended on during the latter part of April and into July, the run-off until June being superabundant for all purposes, but in July it begins to fall below the necessities of dependent industries, and it remains below until the autumnal rains occur. Snow storage has been made a subject of extended observation by Mr. W. F. Englebright, chief engineer of the South Yuba Canal Company, through whose courtesy the writer has been enabled to prepare a most instructive diagram of the accumulation, depth, and rate of melting of snow at Lake Fordyce (fig. 15). This lake has an elevation of 6,500 feet above tide level, and is in a region over which the annual precipitation in rain and melted snow is 70 inches. Snow begins to accumulate late in November, and reaches its maximum depth in packed snow in March. During the winter months the lower readings on the gage rod following higher readings generally indicate a packing of the snow.

[1] These figures are taken from a map prepared, under the writer's direction, for the California Water and Forest Association, showing the drainage areas, the mean annual rainfall, and the distribution of forests throughout the State.

A. CANYON OF MIDDLE FORK OF YUBA RIVER.

B. MIDDLE FORK OF YUBA RIVER NEAR FREEMAN'S CROSSING

Melting begins in March and continues quite regularly until the middle of June or early July. Short storms during April and May cause offsets in the curve, which resumes a parallel line. A series of cold and heavy storms in April, 1896, caused the snow to last until July 5, while the clear, warm spring of 1897 caused it to disappear on June 7.

During the latter half of April, by means of daily reports by telephone, Mr. Englebright is enabled, through diagrams upon a larger scale, to approximate to within a few days the duration of the snow supply and the beginning of the draft on the reservoirs. Data and studies of this kind are very valuable, and suggest the importance of stations above the snow line as a means of determining the volume of snow storage available at different seasons and the ratio between the volume stored by snow and that stored by reservoirs. The discharge of the streams is maintained by snow during the spring and for half of

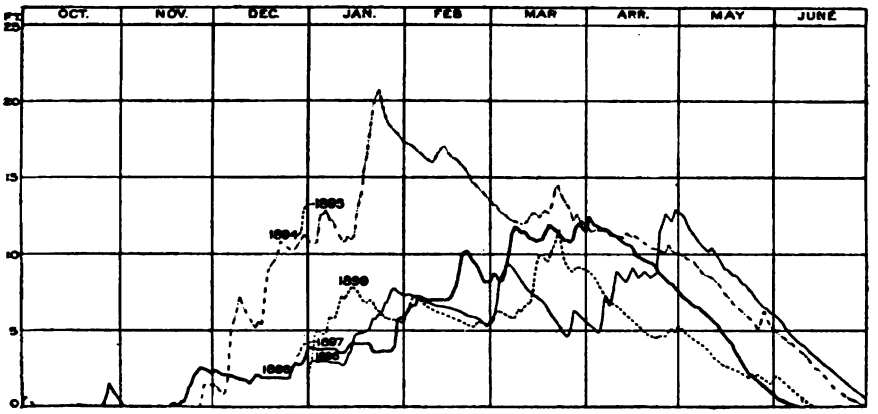

FIG. 15.—Diagram showing depth of snow at Lake Fordyce.

the summer months, and the volume and rate of the discharge can be foretold with reasonable accuracy by daily readings of gages properly located.

ARTIFICIAL STORAGE POSSIBILITIES.

The natural facilities for the storage of storm waters are particularly favorable in the upper third of the drainage basin of the South Fork. The demands for large volumes of water under high pressure to operate the mines in the middle and lower portions of that drainage basin and those on Bear and American rivers were met by the construction of large and expensive canals and storage reservoirs. Just above the great bend north of the head of Bear River and at the head of the steep canyon of the South Fork is a broad, flat, glacial valley which has been converted into a lake by the construction of a stone dam. This lake, known as Lake Spaulding and shown in Pl. IV, *A*, has a capacity of 270,000,000 cubic feet, and is the lower and controlling reservoir of a series embracing the available storage and

supply above. This supply is derived from about 120 square miles, upon which the mean annual precipitation in rain and melted snow is about 5 feet. The following is a list of the storage reservoirs:

Storage reservoirs in Yuba River Basin.

Name of reservoir.	Elevation.	Area.	Capacity.	Cost of dam.
	Feet.	*Acres.*	*Gallons.*	
Meadow	7,515	300	1,275,000,000	$75,000
Stirling	7,200	100	340,000,000	20,000
White Rock	7,000	80	225,000,000	5,000
Peak Lakes (three)	6,900	150	1,275,000,000	(a)
Fordyce	6,500	474	5,950,000,000	300,000
Lost River	7,000	(a)	85,000,000	(a)
Fallcreek Lakes (six)	7,000	171	1,020,000,000	(a)
Spaulding	4,846	215	2,125,000,000	50,000
Summit Lake	6,800	400	1,988,816,000	20,000
Bear Valley	4,400	60	145,411,200	8,000
Total		1,950	b 14,409,227,200	478,000

a Records lost. b 1,921,230,293 cubic feet.

The aggregate area of these reservoirs is 3.05 square miles, and they are filled to an average depth of 22.5 feet, thus giving storage for about 12 per cent of the mean annual precipitation upon the tributary area, the remainder going to waste and to swell the floods which devastate the valley. It is possible, by raising the dams and enlarging the canals, to utilize a considerable additional portion of the precipitation. The conditions favorable to the conservation of water on the upper third of the drainage basin of the South Fork are far greater than in the lower two-thirds of that basin or in the basin of the other forks.

On the upper portion of Canyon Creek, a tributary of the South Fork, the Eureka Lake and Yuba Canal Company and the North Bloomfield Gravel and Mining Company have the following storage reservoirs:

Storage reservoirs on upper portion of Canyon Creek.

Name of reservoir.	Area.	Height.	Top length.	Barometric elevation.	Catchment area.	Capacity.	Cost.
	Acres.	*Feet.*	*Feet.*	*Feet.*	*Acres.*	*Cubic feet.*	
Bowman	500.0	100.0	425	5,450	12,093	930,000,000	$151,521
Sawmill Flat	80.6	39.2		5,780		2,000,000	Total amount expended on these dams and reservoirs, $946,000.
Shot Gun Lake	26.2	10.0		6,410		3,423,816	
Island Lake	48.8	12.8		6,690		23,027,558	
Middle Lake	11.2	12.0		6,460		2,395,800	
Crooked Lake	10.3	3.0		6,510		1,600,000	
Round Lake	8.1	11.0		6,590		2,906,630	
Fall Creek				6,690			
Jackson Lake	20.0	5.0		5,410		15,000,000	
Faucherie Lake	90.0	21.0	550	6,060	3,262	58,000,000	8,000
Weaver Lake	83.5	21.8				150,000,000	
Eureka Lake	337.3	68.2	250	6,480	3,170	661,000,000	35,000
Total						1,849,354,804	

The drainage area tributary to these reservoirs is 28.4 square miles, and it receives a total precipitation during an average year of

A. ENGLISH RESERVOIR ON MIDDLE FORK OF YUBA RIVER.

B SMALL DAM OF ENGLISH RESERVOIR.

4,589,481,600 cubic feet, 1,849,354,804 cubic feet of which is stored, or between one-third and one-half of the mean annual precipitation.

On the Middle Fork there are no reservoirs storing water at the present time. The only site of any importance is that of the Ruyard or English reservoir (see Pl. VI), which has not been in use since the failure of the dam in June, 1883. This site has a capacity of 650,000,000 cubic feet. Weaver Lake is on the watershed of the Middle Yuba, but its catchment area is not large enough to fill it, so it is supplied from the Eureka Lake and Yuba Canal Company's ditch from Canyon Creek, a tributary of the South Yuba, and is included in the foregoing list. At Milton there is a reservoir site with an estimated capacity of 28,000,000 cubic feet. The total storage capacity on the Middle Yuba may be considered to be 678,000,000 cubic feet.

On the North Fork are the dam and headworks of the Browns Valley Irrigation District. (See Pl. VII.) The dam is a well-built crib structure, about 167 feet long on the crest, with a maximum height of 37 feet. The head gates are in concrete. For several miles above the dam the river bed is covered with gravel, sand, and cobbles on a grade slightly less than that of the original stream. This is a feature common to dams upon streams carrying mining débris. The dam thus acts as a retaining wall as well as an overflow weir. Leakage through the débris and dam is slight. The head gates open into a flume 5 feet by 7 feet, on a grade of 13 feet to the mile, and built to carry 300 second-feet of water. The greater portion of the water diverted is used to develop power at the Colgate and Browns Valley power stations of the Bay Counties Power Company, lessees of the Browns Valley Irrigation District rights. This power is transmitted to Marysville, Oroville, Wheatland, Nevada City, Grass Valley, and Sacramento. These plants are synchronized with one on the South Yuba, about 7 miles distant.

On the North Fork there are the following small lakes, which might be developed to an aggregate storage capacity of 500,000,000 cubic feet:

Reservoir sites on the North Fork.

Name of lake.	Area.
	Acres.
Upper Sardine	38.0
Lower Sardine	40.0
Young American	9.0
Volcano	2.5
Packer	7.0
Saxonia	2.5
Deer	5.0
Upper Salmon	30.0
Lower Salmon	50.0
Hawley	11.0
Spencer Lakes (two)	16.0
Sundry small lakes (five, not named)	27.0
Deadmans	3.0
Total	241.0

Assuming that artificial storage on the North Fork and the Middle Fork could be developed to a capacity equal to that above the Lake Spaulding dam, there would then be in service an area of 6.8 square miles with water at an average depth of 26.4 feet, or 5,692,000,000 cubic feet. The mean annual precipitation in the drainage basin of Yuba River is 170,829,000,000 cubic feet. The total ultimate artificial storage is less than 3.3 per cent of this precipitation, and could hardly be recognized in a gaging of the total run-off. Moreover, in the storage of water for industrial purposes the uncertainty of the character of the seasonal rainfall makes it prudent and desirable to permit the reservoirs to fill during the earlier rains, and not leave the husbanding of a supply to possible succeeding rains. Hence it generally happens that when the heavy storms of the late winter and spring months occur the reservoirs are full and the flood wave passes down without being diminished by the reservoirs. This is also true to a limited extent of regions above the snow line, where the unmelted snow constitutes a reservoir of far greater capacity than ordinarily is obtained by building dams. It happens that when late warm rains or rapid melting of the snows occurs the reservoirs are already full, and consequently do not diminish the flood volume.

It would appear, therefore, that however useful artificial reservoirs are for domestic and industrial purposes they can not be relied upon, except under unusual conditions, to decrease the heights of late winter and spring floods, and we must look elsewhere for a solution of that problem.

COMPARISON OF LOW-WATER DISCHARGE FROM A TIMBERED AREA WITH THAT FROM A COMPARATIVELY TREELESS AREA.

On the south fork of the North Fork is a watershed area of 139 square miles, which was gaged on September 19, 1900, after three successive seasons of deficient rainfall, and gave a minimum run-off of 113 second-feet, or 0.8 second-foot per square mile. This area is well covered with timber and brush, and in 120 days it gives a minimum run-off of 1,441,152,000 cubic feet.

The drainage basin of the North Fork is more heavily timbered than the basins of the other forks, and consequently it has a deeper soil, and, although only one-tenth of the total drainage area, it furnishes 75 per cent of the low-water flow of the entire drainage basin above Parks Bar.

On the south fork above Lake Spaulding there is a watershed of 120 square miles, which has heretofore been described as comparatively barren of timber, the timbered areas which once existed having been denuded. (See Pl. VIII, *A*.) The run-off of this area is practically nothing for 120 days of the year, due to the absence of forests and

A. BROWNS VALLEY IRRIGATION DISTRICT DAM FROM ABOVE

B. BROWNS VALLEY IRRIGATION DISTRICT DAM FROM BELOW

brush. If this area were afforested and gave a minimum run-off of 0.8 second-foot per square mile, the discharge would be 100 second-feet, equivalent to an effective storage capacity of 1,036,800,000 cubic feet. This minimum low-water discharge of 100 second-feet for 120 days is equivalent to more than half the storage capacity of all the reservoirs above Lake Spaulding dam, which aggregate 1,375,000,000 cubic feet. As the basis of this estimate is extreme low-water discharge, it may be assumed that by afforesting the watershed this costly and extensive system of reservoirs could safely be drawn upon for double their present capacity. As what is true of portions of the watershed is true of the watershed as a whole, aggregating as it does 1,357 square miles, the value of afforesting the area becomes apparent.

It appears to the writer that the solution of the problem of storage of flood waters is not the retention of a small percentage of the storm waters behind dams, but the application of storage over the entire watershed by the systematic protection and extension of forest-covered and brush-covered areas.

DISCHARGE MEASUREMENTS OF YUBA RIVER AND ITS TRIBUTARIES.

The accompanying tables of low-water discharge measurements of Yuba River and its forks are based upon observations and gagings

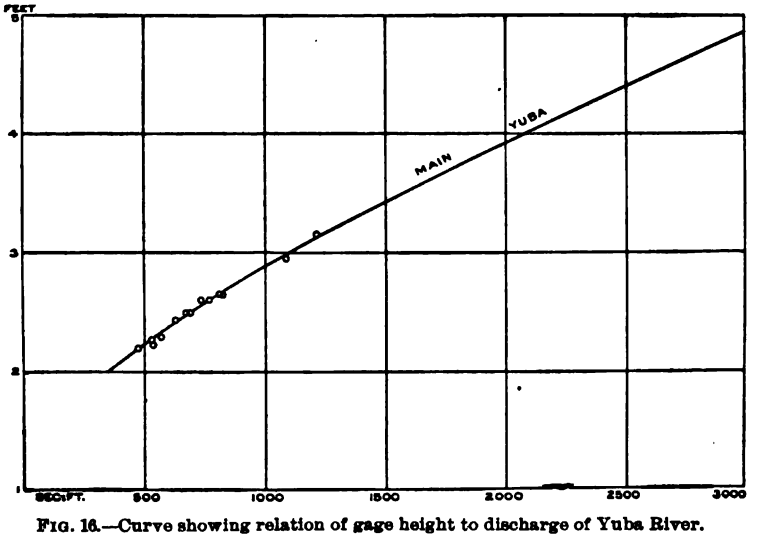

FIG. 16.—Curve showing relation of gage height to discharge of Yuba River.

made by Mr. H. D. H. Connick, under the direction of the writer, during the months of June, July, August, September, and October, 1900. The precipitation during the wet season preceding these gagings was about two-thirds to three-fourths of the mean annual rain-

fall. The precipitation during the two preceding wet seasons was still further below the normal.

The location of the gaging stations is shown on the map, fig. 14, page 40. The gagings were made with a large Price meter furnished by the United States Geological Survey. The usual method was pursued, namely, the cross section was divided into subsections of 5 feet each, and the velocities were observed for three minutes for each foot of depth. The integral of the discharges thus ascertained divided by the total cross-sectional area determined the mean velocity. The volumes thus determined for various stages were platted as a curve of discharge, using gage heights and discharges as ordinates, from which curve intermediate discharges were estimated to fill out the tables. For discharge curves, see figs. 16, 17, and 18.

Daily discharge of Yuba River at Parks Bar Bridge during the month of July, 1900.

Day.	Gage height.		Mean gage height.	Area of section.	Mean velocity.	Discharge.	
	6 a.m.	6 p.m.					
	Feet.	*Feet.*	*Feet.*	*Square feet.*	*Feet per second.*	*Second-feet.*	*Cubic feet per 24 hours.*
1	2.90	2.80	2.85			970.0	83,808,000
2	2.90	2.90	2.90			1,020.0	88,128,000
3	3.10	2.90	3.00			1,100.0	95,040,000
4	2.80	2.70	2.75			890.0	76,896,000
5	2.70	2.80	2.75			890.0	76,896,000
6	2.60	2.70	2.65			810.0	69,984,000
7	2.60	2.80	2.70			850.0	73,440,000
8	2.70	2.70	2.70			850.0	73,440,000
9	2.60	2.60	2.60			775.0	66,960,000
10	2.60	2.60	2.60			775.0	66,960,000
11	2.70	2.60	2.65	181.5	4.52	820.8	70,917,120
12	2.70	2.50	2.60	180.0	4.50	810.5	70,027,200
13	2.70	2.50	2.60	171.1	4.39	773.8	66,856,320
14	2.60	2.50	2.55			730.0	63,072,000
15	2.60	2.50	2.55			730.0	63,072,000
16	2.50	2.40	2.45			660.0	57,024,000
17	2.50	2.40	2.45			660.0	57,024,000
18	2.50	2.40	2.45			660.0	57,024,000
19	2.40	2.40	2.40			620.0	53,568,000
20	2.40	2.30	2.35			580.0	50,112,000
21	2.40	2.30	2.35			580.0	50,112,000
22	2.40	2.40	2.40	163.6	3.80	622.2	53,758,080
23	2.40	2.50	2.45	173.3	3.91	685.8	59,253,120
24	2.50	2.40	2.45	169.3	3.97	672.7	57,625,280
25	2.50	2.40	2.45			660.0	57,024,000
26	2.40	2.30	2.35			580.0	50,112,000
27	2.40	2.30	2.35			580.0	50,112,000
28	2.40	2.30	2.35			580.0	50,112,000
29	2.40	2.30	2.35			580.0	50,112,000
30	2.30	2.40	2.35			580.0	50,112,000
31	2.30	2.30	2.30			545.0	47,088,000
Total							1,955,059,000

A. SECOND-GROWTH FOREST DESTROYED BY FIRE.

B. HEAD OF SOUTH YUBA CANAL COMPANY'S DITCH.

Daily discharge of Yuba River at Parks Bar Bridge during the month of August, 1900.

Day.	Gage height.		Mean gage height.	Area of section.	Mean velocity.	Discharge.	
	6 a. m.	6 p. m.					
	Feet.	*Feet.*	*Feet.*	*Sq. ft.*	*Ft. per sec.*	*Sec.-ft.*	*Cu. ft. per 24 hrs.*
1	2.30	2.30	2.30			545.0	47,088,000
2	2.20	2.30	2.25			510.0	44,064,000
3	2.20	2.20	2.20			480.0	41,472,000
4	2.20	2.20	2.20			480.0	41,472,000
5	2.20	2.20	2.20			480.0	41,472,000
6	2.20	2.20	2.20			480.0	41,472,000
7	2.30	2.20	2.25	151.8	3.54	536.8	46,379,520
8	2.30	2.20	2.25	158.7	3.59	571.2	49,251,680
9	2.30	2.20	2.25			510.0	44,064,000
10	2.20	2.10	2.15			440.0	38,016,000
11	2.20	2.20	2.20			480.0	41,472,000
12	2.10	2.20	2.15			440.0	38,016,000
13	2.20	2.30	2.25			510.0	44,064,000
14	2.20	2.30	2.25	153.0	3.49	535.0	46,094,400
15	2.10	2.20	2.15			440.0	38,016,000
16	2.20	2.20	2.20			480.0	41,472,000
17	2.10	2.10	2.10			400.0	34,560,000
18	2.10	2.20	2.15			440.0	38,016,000
19	2.10	2.20	2.15			440.0	38,016,000
20	2.30	2.20	2.25			510.0	44,064,000
21	2.30	2.30	2.30			545.0	47,088,000
22	2.40	2.30	2.35			580.0	50,112,000
23	2.30	2.20	2.25			510.0	44,064,000
24	2.30	2.20	2.25			510.0	44,064,000
25	2.20	2.10	2.15			440.0	38,016,000
26	2.10	2.10	2.10			400.0	34,560,000
27	2.20	2.10	2.10			400.0	34,560,000
28	2.10	2.10	2.10			474.2	40,970,880
29	2.10	2.10	2.10			400.0	34,560,000
30	2.20	2.20	2.20			480.0	41,472,000
31	2.20	2.10	2.15			440.0	38,016,000
Total							1,286,151,520

Daily discharge of Yuba River at Parks Bar Bridge during the month of September, 1900.

Day.	Gage height.		Mean gage height.	Area of section.	Mean velocity.	Discharge.	
	6 a. m.	6 p. m.					
	Feet.	*Feet.*	*Feet.*	*Sq. ft.*	*Ft. per sec.*	*Sec.-ft.*	*Cu. ft. per 24 hrs.*
1	2.10	2.10	2.10			400.0	34,560,000
2	2.10	2.10	2.10			400.0	34,560,000
3	2.10	2.10	2.10			400.0	34,560,000
4	2.10	2.20	2.15			440.0	38,016,000
5	2.40	2.40	2.40			620.0	53,568,000
6	2.30	2.30	2.30			545.0	47,088,000
7	2.20	2.20	2.20			470.0	40,608,000
8	2.20	2.20	2.20			470.0	40,608,000
9	2.20	2.10	2.15			440.0	38,016,000
10	2.10	2.10	2.10			400.0	34,560,000
11	2.30	2.20	2.25			510.0	44,064,000
12	2.20	2.30	2.25			510.0	44,064,000
13	2.30	2.50	2.40			620.0	53,568,000
14	2.60	2.60	2.60	178.5	4.12	735.6	63,555,840
15	2.40	2.40	2.40			620.0	53,568,000
16	2.40	2.30	2.35			580.0	50,112,000
17	2.20	2.20	2.20			470.0	40,618,000
18	2.20	2.20	2.20			470.0	40,618,000
19	2.20	2.20	2.20			470.0	40,618,000
20	2.20	2.10	2.15			440.0	38,016,000
21	2.10	2.10	2.10			400.0	34,560,000
22	2.10	2.10	2.10			400.0	34,560,000
23	2.00	2.00	2.00			360.0	31,104,000
24	2.00	2.00	2.00			360.0	31,104,000
25	2.00	2.00	2.00			360.0	31,104,000
26	2.10	2.10	2.10			400.0	34,560,000
27	2.00	2.00	2.00			360.0	31,104,000
28	2.00	2.00	2.00			360.0	31,104,000
29	2.00	2.00	2.00			360.0	31,104,000
30	2.00	2.00	2.00			360.0	31,104,000
Total							1,158,249,840

Daily discharge of Yuba River at Parks Bar Bridge during the month of October, 1900.

Day.	Gage height.		Mean gage height.	Discharge.	
	6 a. m.	6 p. m.			
	Feet.	*Feet.*	*Feet.*	*Sec.-ft.*	*Cu. ft. per 24 hrs.*
1	2.00	2.00	2.00	360	31,104,000
2	2.00	2.00	2.00	360	31,104,000
3	2.80	3.40	3.10	1,200	103,680,000
4	3.20	4.90	4.05	2,140	184,896,000
5	4.60	4.60	4.60	2,720	235,008,000
6	4.20	4.00	4.10	2,200	190,080,000
7	3.70	3.60	3.65	1,730	149,472,000
8	3.50	3.50	3.50	1,595	137,808,000
9	3.50	3.50	3.50	1,595	137,808,000
10	3.50	3.40	3.45	1,540	133,056,000
11	3.40	3.40	3.40	1,480	127,872,000
12	3.60	3.40	3.50	1,595	137,808,000
13	3.40	3.40	3.40	1,480	127,872,000
Total					1,727,568,000

NOTE.—On the evening of October 4 gage No. 1 was washed out. All subsequent readings were made on gage No. 2. Equation: Gage No. 2+2.2=gage No. 1. Thus: 2.7+2.2=4.9, gage height.

Daily discharge of North Fork of Yuba River at Yuba Power Company's dam during the month of July, 1900.

Day.	Gage height.		Mean gage height.	Area of section.	Mean velocity.	Discharge.	
	6 a. m.	6 p. m.					
	Feet.	*Feet.*	*Feet.*	*Sq. ft.*	*Ft. per sec.*	*Sec.-ft.*	*Cu. ft. per 24 hrs*
1	2.07	2.07	2.07			630.0	54,432,000
2	2.06	2.06	2.06			625.0	54,000,000
3	2.05	2.05	2.05	203.1	2.98	606.0	52,358,400
4	2.05	2.00	2.03			610.0	52,704,000
5	2.00	2.00	2.00			595.0	51,408,000
6	1.95	2.00	1.98	196.4	2.88	566.8	48,971,520
7	2.00	2.00	2.00			595.0	51,408,000
8	1.95	1.95	1.95			560.0	48,384,000
9	1.90	1.85	1.88			520.0	44,928,000
10	1.85	1.80	1.83			490.0	42,336,000
11	1.80	1.80	1.80			470.0	40,608,000
12	1.75	1.75	1.75			440.0	38,016,000
13	1.75	1.75	1.75			440.0	38,016,000
14	1.75	1.75	1.75			440.0	38,016,000
15	1.75	1.75	1.75			440.0	38,016,000
16	1.75	1.75	1.75			440.0	38,016,000
17	1.70	1.70	1.70	160.3	2.73	437.7	37,817,280
18	1.70	1.70	1.70	157.0	2.66	418.7	36,175,680
19	1.70	1.70	1.70			410.0	35,424,000
20	1.65	1.65	1.65			380.0	32,832,000
21	1.65	1.65	1.65			380.0	32,832,000
22	1.65	1.70	1.68			400.0	34,560,000
23	1.70	1.70	1.70			410.0	35,424,000
24	1.70	1.65	1.68			400.0	34,560,000
25	1.65	1.65	1.65			380.0	32,832,000
26	1.65	1.65	1.65			380.0	32,832,000
27	1.65	1.65	1.65	158.5	2.31	366.1	31,631,040
28	1.60	1.60	1.60			350.0	30,240,000
29	1.60	1.60	1.60	159.2	2.33	371.4	32,088,960
30	1.60	1.60	1.60			350.0	30,240,000
31	1.60	1.60	1.60	159.1	2.28	363.9	31,440,960
Total							1,232,548,840

Daily discharge of North Fork of Yuba River at Yuba Power Company's dam during the month of August, 1900.

Day.	Gage height.		Mean gage height.	Area of section.	Mean velocity.	Discharge.	
	6 a. m.	6 p. m.					
	Feet.	*Feet.*	*Feet.*	*Sq. ft.*	*Ft. per sec.*	*Sec.-ft.*	*Cu. ft. per 24 hrs.*
1	1.60	1.60	1.60			350.0	30,240,000
2	1.55	1.55	1.55			320.0	27,648,000
3	1.55	1.55	1.55			320.0	27,648,000
4	1.55	1.55	1.55			320.0	27,648,000
5	1.55	1.55	1.55			320.0	27,648,000
6	1.55	1.55	1.55			320.0	27,648,000
7	1.55	1.55	1.55			320.0	27,648,000
8	1.55	1.55	1.55			320.0	27,648,000
9	1.55	1.55	1.55	155.4	2.11	328.4	28,373,760
10	1.55	1.55	1.55	153.0	2.10	321.7	27,794,880
11	1.55	1.55	a1.55			320.0	27,648,000
12	1.55	1.55	1.55			320.0	27,648,000
13	1.55	1.55	1.55			320.0	27,648,000
14	1.55	1.55	1.55			320.0	27,648,000
15	1.55	1.55	1.55			320.0	27,648,000
16	1.55	1.55	1.55			320.0	27,648,000
17	1.55	1.55	1.55			320.0	27,648,000
18	1.55	1.55	1.55			320.0	27,648,000
19	1.50	1.50	1.50			300.0	25,920,000
20	1.50	1.50	1.50			300.0	25,920,000
21	1.50	1.50	1.50			300.0	25,920,000
22	1.50	1.50	1.50			300.0	25,920,000
23	1.50	1.50	1.50			300.0	25,920,000
24	1.50	1.50	1.50			300.0	25,920,000
25	1.50	1.50	1.50			300.0	25,920,000
26	1.50	1.50	1.50			300.0	25,920,000
27	1.49	1.49	1.49			295.0	25,488,000
28	1.49	1.48	1.49			295.0	25,488,000
29	1.48	1.48	1.48	145.8	1.93	282.1	24,373,440
30	1.47	1.47	1.47	142.7	1.99	284.6	24,589,440
31	1.47	1.47	1.47			280.0	24,192,000
Total							814,971,520

Daily discharge of North Fork of Yuba River at Yuba Power Company's dam during the month of September, 1900.

Day.	Gage height.		Mean gage height.	Discharge.	
	6 a. m.	6 p. m.			
	Feet.	*Feet.*	*Feet.*	*Sec.-ft.*	*Cu. ft. per 24 hrs.*
1	1.46	1.46	1.46	270	23,328,000
2	1.46	1.46	1.46	270	23,328,000
3	1.46	1.45	1.46	270	23,328,000
4	1.46	1.47	1.47	280	24,192,000
5	1.48	1.49	1.49	295	25,448,000
6	1.49	1.50	1.50	300	25,920,000
7	1.50	1.50	1.50	300	25,920,000
8	1.49	1.49	1.49	295	25,488,000
9	1.49	1.48	1.49	295	25,488,000
10	1.47	1.47	1.47	280	24,192,000
11	1.47	1.48	1.48	280	24,192,000
12	1.49	1.49	1.49	295	25,488,000
13	1.50	1.56	1.53	310	26,784,000
14	1.61	1.60	1.61	355	30,672,000
15	1.55	1.50	1.53	310	26,784,000
16	1.50	1.50	1.50	300	25,920,000
17	1.49	1.49	1.49	295	25,488,000
18	1.48	1.48	1.48	280	24,192,000
19	1.47	1.47	1.47	280	24,192,000
20	1.47	1.46	1.47	280	24,192,000
21	1.46	1.46	1.46	270	23,328,000
22	1.45	1.45	1.45	266	22,982,400
23	1.44	1.43	1.44	265	22,896,000
24	1.44	1.45	1.45	266	22,982,400
25	1.45	1.46	1.46	270	23,328,000
26	1.47	1.48	1.48	280	24,192,000
27	1.47	1.48	1.48	280	24,192,000
28	1.48	1.47	1.48	280	24,192,000
29	1.47	1.47	1.47	280	24,192,000
30	1.46	1.46			
Total					710,830,800

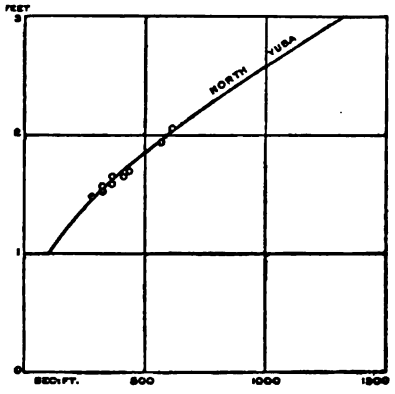

FIG. 17.—Curve showing relation of gage height to discharge of North Fork of Yuba River.

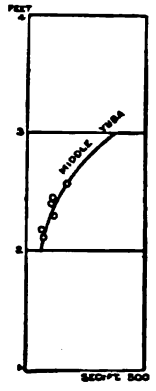

FIG. 18.—Curve showing relation of gage height to discharge of Middle Fork of Yuba River.

Daily discharge of North Fork of Yuba River at Yuba Power Company's dam during the month of October, 1900.

Day.	Gage height.		Mean gage height.	Discharge.	
	6 a. m.	6 p. m.			
	Feet.	*Feet.*	*Feet.*	*Sec.-ft.*	*Cu. ft. per 24 hrs.*
1	1.46	1.47	1.47	280	24,192,000
2	1.48	1.50	1.49	295	25,488,000
3	1.99	2.40	2.19	720	62,208,000
4	1.93	2.15	2.04	610	52,704,000
5	2.68	2.70	2.69	1,080	93,312,000
6	2.12	1.95	2.03	610	52,704,000
7	1.81	1.80	1.80	470	40,608,000
8	1.70	1.68	1.69	405	34,992,000
9	1.67	1.75	1.71	410	35,424,000
10	1.64	1.71	1.68	400	34,560,000
11	1.60	1.70	1.65	350	32,352,000
12	1.64	1.70	1.67	390	33,696,000
13	1.65	1.68	1.67	390	33,696,000
Total					556,416,000

Daily discharge of Middle Fork of Yuba River at Freeman's bridge during the month of July, 1900.

Day.	Gage height.		Mean gage height.	Area of section.	Mean velocity.	Discharge.	
	6 a. m.	6 p. m.					
	Feet.	*Feet.*	*Feet.*	*Sq. ft.*	*Ft. per sec.*	*Sec.-ft.*	*Cu. ft. per 24 hrs.*
1			2.40	63.0	3.06	190.8	16,485,120
2			2.38			189.0	15,329,600
3			2.35	59.9	3.00	179.9	15,543,360
4			2.33	58.1	3.18	184.9	15,975,360
5			2.40			190.0	16,416,000
6	2.45	2.60	2.53			150.0	12,960,000
7	2.60	2.60	2.60	62.6	2.58	161.7	13,970,880
8	2.60	2.45	2.53			150.0	12,960,000
9	2.45	2.45	2.45			130.0	11,232,000
10	2.45	2.40	2.43			125.0	10,810,000
11	2.40	2.40	2.40			120.0	10,368,000
12	2.40	2.40	2.40			120.0	10,368,000
13	2.30	2.30	2.30			100.0	8,640,000
14	2.30	2.30	2.30			100.0	8,640,000
15	2.30	2.30	2.30			100.0	8,640,000
16	2.30	2.40	2.35			110.0	9,504,000
17	2.30	2.40	2.35			110.0	9,504,000
18	2.30	2.35	2.33			108.0	9,331,200
19	2.45	2.40	2.43			125.0	10,810,000
20	2.35	2.35	2.35			110.0	9,504,000
21	2.35	2.40	2.38			119.0	10,281,600
22	2.35	2.40	2.38			119.0	10,281,600
23	2.35	2.45	2.40			120.0	10,368,000
24	2.40	2.35	2.38			119.0	10,281,600
25	2.35	2.40	2.38			119.0	10,281,600
26	2.35	2.35	2.35			110.0	9,504,000
27	2.35	2.40	2.38			119.0	10,281,600
28	2.35	2.34	2.35			110.0	9,504,000
29	2.30	2.35	2.33	49.6	2.20	109.2	9,434,880
30	2.25	2.30	2.28			90.0	7,776,000
31	2.20	2.20	2.20			80.0	6,912,000
Total							341,898,400

Daily discharge of Middle Fork of Yuba River at Freeman's bridge during the month of August, 1900.

Day.	Gage height.		Mean gage height.	Area of section.	Mean velocity.	Discharge.	
	6 a. m.	6 p. m.					
	Feet.	*Feet.*	*Feet.*	*Sq. ft.*	*Ft. per sec.*	*Sec.-ft.*	*Cu. ft. per 24 hrs.*
1	2.20	2.20	2.20			80.0	6,912,000
2	2.15	2.20	2.18			79.0	6,825,600
3	2.15	2.15	2.15			75.0	6,480,000
4	2.15	2.15	2.15			75.0	6,480,000
5	2.15	2.15	2.15			75.0	6,480,000
6	2.15	2.15	2.15			75.0	6,480,000
7	2.15	2.15	2.15			75.0	6,480,000
8	2.15	2.15	2.15			75.0	6,480,000
9	2.15	2.15	2.15			75.0	6,480,000
10	2.15	2.15	2.15			75.0	6,480,000
11	2.15	2.15	2.15	36.5	2.17	79.3	6,851,520
12	2.15	2.15	2.15	36.7	2.13	78.1	6,747,840
13	2.15	2.15	2.15			75.0	6,480,000
14	2.15	2.15	2.15			75.0	6,480,000
15	2.15	2.15	2.15			75.0	6,480,000
16	2.15	2.15	2.15			75.0	6,480,000
17	2.15	2.15	2.15			75.0	6,480,000
18	2.15	2.15	2.15			75.0	6,480,000
19	2.15	2.15	2.15			75.0	6,480,000
20	2.15	2.15	2.15			75.0	6,480,000
21	2.15	2.15	2.15			75.0	6,480,000
22	2.15	2.15	2.15			75.0	6,480,000
23	2.15	2.15	2.15			75.0	6,480,000
24	2.15	2.15	2.15			70.0	6,048,000
25	2.10	2.10	2.10			70.0	6,048,000
26	2.10	2.10	2.10			70.0	6,048,000
27	2.10	2.10	2.10			70.0	6,048,000
28	2.10	2.10	2.10			70.0	6,048,000
29	2.10	2.10	2.10	31.2	2.22	69.26	5,984,064
30	2.10	2.10	2.10	30.5	2.22	67.7	5,849,280
31	2.10	2.10	2.10				
Total							192,530,304

Daily discharge of Middle Fork of Yuba River at Freeman's bridge during the month of September, 1900.

Day.	Gage height.		Mean gage height.	Area of section.	Mean velocity.	Discharge.	
	6 a. m.	6 p. m.					
	Feet.	*Feet.*	*Feet.*	*Sq. ft.*	*Ft. per sec.*	*Sec.-ft.*	*Cu. ft. per 24 hrs.*
1	2.10	2.10	2.10			70.0	6,048,000
2	2.10	2.05	2.08			68.0	5,875,000
3	2.00	2.00	2.00			60.0	5,184,000
4	2.00	2.00	2.00			60.0	5,184,000
5	2.05	2.05	2.05			65.0	5,616,000
6	2.05	2.05	2.05				5,616,000
7	2.05	2.10	2.08			68.0	5,875,000
8	2.05	2.05	2.05			65.0	5,616,000
9	2.05	2.05	2.05			65.0	5,616,000
10	2.00	2.00	2.00			60.0	5,184,000
11	2.00	2.05	2.03			62.0	5,356,800
12	2.05	2.05	2.05			65.0	5,616,000
13	2.15	2.35	2.25			90.0	7,776,000
14	2.30	2.30	2.30			100.0	8,640,000
15	2.20	2.20	2.20			80.0	6,912,000
16	2.20	2.20	2.20			80.0	6,912,000
17	2.15	2.15	2.15			75.0	6,480,000
18	2.15	2.15	2.15	27.4	2.34	64.2	5,546,880
19	2.15	2.15	2.15			75.0	6,480,000
20	2.10	2.10	2.10			70.0	6,048,000
21	2.00	2.00	2.00			60.0	5,184,000
22	2.00	2.00	2.00			60.0	5,184,000
23	2.00	2.00	2.00			60.0	5,184,000
24	2.00	2.00	2.00			60.0	5,184,000
25	2.00	2.00	2.00			60.0	5,184,000
26	2.00	2.00	2.00			60.0	5,184,000
27	2.00	2.00	2.00			60.0	5,184,000
28	2.00	2.00	2.00			60.0	5,184,000
29	2.00	2.00	2.00			60.0	5,184,000
30	2.00	2.00	2.00			60.0	5,184,000
Total							173,421,680

Discharge of Middle Fork of Yuba River at Freeman's bridge during the month of October, 1900.

Day.	Gage height.		Mean gage height.	Discharge.	
	6 a. m.	6 p. m.			
	Feet.	*Feet.*	*Feet.*	*Sec.-ft.*	*Cu. ft. per 24 hrs.*
1	2.00	2.00	2.00	60	5,184,000
2	2.00	2.00	2.00	00	5,184,000
3	2.45	2.45	2.45	130	11,232,000
4	2.40	2.50	2.45	130	11,232,000
5	3.00	3.00	3.00	360	21,104,000
6	2.00	2.50	2.25	90	7,776,000
7	2.45	2.35	2.40	120	10,368,000
8	2.30	2.20	2.25	90	7,776,000
9	2.20	2.20	2.20	80	6,912,000
10	2.15	2.20	2.17	78	6,739,200
11	2.00	2.15	2.08	68	5,875,000
12	2.00	2.15	2.08	68	5,875,000
13	2.00	2.00	2.00	60	5,184,000
Total					110,441,200

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DEPARTMENT OF THE INTERIOR

WATER-SUPPLY

AND

IRRIGATION PAPERS

OF THE

UNITED STATES GEOLOGICAL SURVEY

No. 47

WASHINGTON
GOVERNMENT PRINTING OFFICE
1901

UNITED STATES GEOLOGICAL SURVEY

CHARLES D. WALCOTT, DIRECTOR

OPERATIONS AT RIVER STATIONS, 1900

A REPORT OF THE

DIVISION OF HYDROGRAPHY

OF THE

UNITED STATES GEOLOGICAL SURVEY

PART I

WASHINGTON
GOVERNMENT PRINTING OFFICE
1901

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LETTER OF TRANSMITTAL.

DEPARTMENT OF THE INTERIOR,
UNITED STATES GEOLOGICAL SURVEY,
DIVISION OF HYDROGRAPHY,
Washington, D. C., March 1, 1901.

SIR: I have the honor to transmit herewith a manuscript giving the results of operations at various river stations in 1900, and request that it be published in the series of Water-Supply and Irrigation Papers. In order to comply with the law limiting these papers to 100 pages, it is necessary to divide the material. This has been done n a general geographic basis, following the precedent of Water-Supply Papers Nos. 35 to 39, relating to similar operations for the year 1899. In the first paper are introductory remarks and the data relating to New England and Eastern streams. The succeeding papers take up in order the rivers flowing into the Atlantic Ocean, then those tributary to the Ohio and Mississippi rivers, the Great Lakes, Missouri River and the Rio Grande, the interior basin, and finally those flowing into the Pacific Ocean, from north to south, ending with the data relating to the streams of southern California.

The general conclusions drawn from the operations at the river stations, together with diagrams, maps, and illustrations, are being prepared for publication in Part IV of the Twenty-second Annual Report, this being designed to be in form and substance similar to preceding volumes known as Part IV of the Eighteenth to the Twenty-first Annual Reports.

Very respectfully,

F. H. NEWELL,
Hydrographer in Charge.

Hon. CHARLES D. WALCOTT,
Director United States Geological Survey.

OPERATIONS AT RIVER STATIONS, 1900.

PART I.

INTRODUCTION.

The following pages contain descriptions of the river stations maintained during 1900 by the United States Geological Survey, together with details of the average daily height of the water and results of measurements of discharge. The rating tables constructed from the latter and applicable in general for the calendar year will be given at the end of the publication. Similar facts have been printed for the year 1899 in Water-Supply Papers Nos. 35 to 39, inclusive, and a general description of the method of publication has been given on page 9 of Paper No. 35.

ACKNOWLEDGMENTS.

Most of the measurements herewith presented have been obtained through local hydrographers, a comparatively small part of the work having been conducted directly from the office of the Geological Survey at Washington. Acknowledgment is therefore due to these persons individually. Thanks should also be extended to individuals and corporations who have cooperated in various ways—by furnishing readings of the heights of water, by assisting in transportation, etc. The following list gives the names of the resident hydrographers or persons cooperating, arranged alphabetically by States:

California: J. B. Lippincott, civil engineer, Los Angeles.

Colorado: A. L. Fellows, civil engineer, Denver.

Georgia and Alabama: Prof. B. M. Hall, civil engineer, Atlanta; Prof. W. S. Yeates, State geologist, Atlanta; and Prof. Eugene A. Smith, State geologist, Tuscaloosa.

Idaho: N. S. Dils, civil engineer, Caldwell.

Kansas: W. G. Russell, Russell.

Maryland: Prof. W. B. Clark, State geologist, Baltimore.

Montana: Prof. Samuel Fortier, Bozeman; Prof. Fred D. Smith, Missoula.

Nebraska: Prof. O. V. P. Stout, State University, Lincoln, assisted by Adna Dobson and Glenn E. Smith.

Nevada: L. H. Taylor, civil engineer, Golconda.

New Mexico: P. E. Harroun, civil engineer, Albuquerque.

North and South Carolina: Prof. J. A. Holmes, State geologist, Chapel Hill, North Carolina, assisted by E. W. Myers.

Texas: Prof. Thomas U. Taylor, State University

Utah: Prof. George L. Swendsen, Logan.

Virginia and West Virginia: Prof. D. C. Humphreys, Washington and Lee University, Lexington, Virginia.

Washington: Sydney Arnold, civil engineer, North Yakima; William J. Ware, civil engineer, Port Angeles.

Wyoming: A. J. Parshall, civil engineer, Cheyenne.

In a number of instances related data have been inserted, such as results of computation of daily flow at milldams made by local engineers and data of river heights obtained from the United States Weather Bureau or the Corps of Engineers, United States Army. Reference to these facts, mainly unpublished, has been or will be made in other publications of this Survey, and they are therefore placed in consecutive order for convenience of reference.

The methods of measuring the discharge of various streams and of preparing the computations have been described on pages 18 to 30 of the Nineteenth Annual Report, Part IV, and on pages 20 to 22 of the Twentieth Annual Report, Part IV.

METHOD OF USING STREAM GAGINGS FOR THE COMPUTATION OF WATER POWER.[1]

One of the objects of the gagings which are made by the United States Geological Survey is to assist engineers in the estimation of available power on the larger streams of the United States. In an article by John W. Hays in the Manufacturers Record of January 10, 1901, there is given a description of the use which he has made of the daily records published by the Survey. The suggestions made by Mr. Hays are of such general interest that they are given herewith.

Speaking generally, to determine the power available on any stream it is necessary to know the fall and the flow. The fall can easily be determined by an engineer, but the flow of many of our streams is still very uncertain. Usually, an engineer, after obtaining all available data of the stream being studied and also of the neighboring streams, must rely largely upon his judgment to determine maximum and minimum flows and the power that can economically be developed. Often the only basis of estimate is the knowledge of what some other stream has been doing within a limited period, or a comparison with some power on the same stream. When it is remembered that during extreme low water a variation in level of only a few inches may affect the quantity of water passing as much as 100 per cent, it will be seen how misleading may be a single gage or float measurement, even when reinforced by the assurance of casual observers that "the river is as low as it ever gets." Yet it is upon such data that many estimates of prospective power have been made, mills built, and machinery installed, only to have the first dry season show conclusively that the estimates were greatly in error. Then has followed the raising of the dam for storage, which has rarely met expectations, and, after many

[1] Report of H. A. Pressey.

shut-downs, the inevitable steam auxiliary comes, and with it regret, on the part of the owners, that hydraulic development was attempted.

The method described by Mr. Hays in the article referred to has been used in computations made from time to time of the available power of the Potomac and other streams, and found to be reliable and convenient. Mr. Hays illustrates the method by quoting the record of Deep River, in North Carolina, as published by the Geological Survey. The various stages of that river for 1899, as read on the gage of the Survey, are given in the following table:

Daily gage height, in feet, of Deep River at Moncure, North Carolina, for 1899.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	2.03	4.58	6.91	7.12	2.99	2.84	1.52	4.54	0.85	0.83	12.74	2.77
2	3.21	5.00	6.25	4.70	2.85	2.61	1.36	4.36	.79	.78	5.86	1.85
3	3.41	6.78	6.39	4.52	2.56	3.45	1.34	3.46	.75	.88	4.23	1.83
4	3.27	7.06	13.25	5.16	2.51	3.13	1.18	3.08	.91	.79	3.48	2.19
5	2.79	16.40	14.43	8.64	2.29	2.49	4.20	1.46	.73	.87	3.04	2.15
6	2.47	20.48	15.77	6.18	2.25	1.95	2.42	1.20	.71	7.20	2.74	1.49
7	10.27	23.21	15.67	6.40	2.47	1.65	2.72	4.54	.89	6.49	2.24	1.27
8	9.13	25.92	13.62	17.76	8.85	1.49	2.11	4.11	1.19	6.70	1.91	1.19
9	7.71	20.90	10.61	15.04	6.09	1.58	2.19	4.15	1.10	7.11	1.86	1.11
10	5.59	18.70	7.39	8.28	4.43	2.55	2.10	3.37	1.82	3.96	.84	1.19
11	4.99	12.26	5.05	6.16	3.85	3.23	1.79	1.39	2.10	3.19	.89	4.03
12	5.83	7.19	4.41	5.40	5.89	4.72	1.67	1.41	2.55	1.97	1.09	5.95
13	6.07	6.98	4.67	4.86	7.99	3.78	1.34	1.25	2.04	1.72	1.02	4.69
14	9.79	6.06	3.91	4.48	9.23	3.40	1.19	1.21	4.86	1.59	1.09	3.39
15	13.07	5.10	21.19	3.76	5.99	2.56	1.09	1.29	1.96	1.31	.96	2.35
16	8.61	6.76	22.51	3.98	4.46	2.42	1.28	1.31	1.04	1.17	.98	1.39
17	7.91	16.20	16.21	4.10	3.97	2.40	.90	1.17	.93	1.27	1.08	1.34
18	6.93	21.26	9.99	4.04	3.75	1.68	1.14	1.09	.84	1.04	1.26	1.47
19	5.99	19.34	14.69	3.54	3.69	1.58	1.11	1.25	.87	1.06	1.14	1.39
20	4.73	15.30	15.31	3.32	3.05	1.49	1.02	.88	1.71	1.12	1.02	.63
21	3.15	9.92	9.02	3.46	2.89	1.50	1.02	.74	1.99	.97	.98	1.27
22	3.36	9.05	7.15	2.97	2.67	1.47	1.02	.78	1.49	.90	1.10	1.13
23	3.08	8.22	5.87	3.10	3.25	1.77	1.19	.90	1.17	.87	1.05	1.07
24	3.17	7.92	5.43	2.96	2.85	1.37	1.11	.95	.87	.85	1.04	1.19
25	3.25	6.24	4.91	2.97	2.75	1.55	1.52	1.06	2.34	.91	1.00	1.13
26	3.27	6.92	4.41	5.60	2.51	1.57	1.67	.84	1.53	.92	1.32	1.11
27	3.54	12.94	4.14	5.54	2.25	1.71	5.19	1.00	1.35	1.01	1.92	1.07
28	3.19	18.22	7.07	4.46	2.02	2.15	4.35	.96	1.09	.98	1.83	.95
29	2.55	.....	11.65	3.50	1.91	1.68	3.77	1.20	1.17	.94	3.78	1.03
30	2.21	.....	10.31	3.04	2.54	1.60	2.11	1.22	1.04	.89	2.84	.97
31	3.23	.....	8.44	.....	2.96	.....	6.28	.89	.....	1.13	.....	1.03

As will be seen from an inspection of the foregoing table, the records kept by the Geological Survey show the stage of water of each stream measured on each day of the year, and the rating table, which is computed from numerous measurements, shows the quantity of water passing at each reading of the gage. These data may most conveniently be applied by developing a table as follows:

In the first column arrange in order all graduations of the gage rod from the lowest to the highest. In the succeeding columns write opposite each gage height the following: (*a*) The number of days during the year on which the gage read that particular height; (*b*) the equivalent volume in second-feet; (*c*) the effective head; (*d*) the equivalent gross horsepower; (*e*) the number of days during the year on which power would have been lessened by low water and by high water, respectively. By way of illustration: The gage read 1.6 feet on 17 days, equivalent to a volume of 380 second-feet, an effective head of 46.4 feet, and a gross horsepower of 2,004, which would be lessened

on 121 days by deficient water and on 227 days by high water in the tailrace. If the gage readings at Moncure station from the records of the year 1899 be thus assembled and the variation of the effective head be noted, there results the following table:

Table showing method of determining the daily available horsepower of a stream from records kept by the United States Geological Survey, using the gage heights of Deep River at Moncure, North Carolina, for the year 1899 as a basis.

Gage height.	Number of days during year 1899 that gage stood at respective heights.	Equivalent volume.	Equivalent effective head.	Equivalent gross horsepower.	Number of days on which power indicated would have been diminished.			Ratio of diminution to total annual power.		
					By low water.	By high water.	Total.	Low water.	High water.	Total.
Feet.		*Sec.-ft.*	*Feet.*					*Per ct.*	*Per ct.*	*Per ct.*
0.6	1	210	47.4	1,128	0	364	364	0.000	0.077	0.07
0.8	27	230	47.2	1,232	1	337	338	.010	.075	.08
1.0	48	250	47.0	1,335	28	289	317	.022	.072	.09
1.2	27	290	46.8	1,544	76	262	338	.045	.067	.11
1.4	18	330	46.6	1,747	103	244	347	.065	.065	.13
1.6	17	380	46.4	2,004	121	227	348	.095	.058	.15
1.8	9	440	46.2	2,280	138	218	356	.125	.055	.18
2.0	13	500	46.0	2,530	147	205	352	.155	.055	.21
2.2	8	560	45.8	2,889	160	197	357	.189	.055	.24
2.4	9	630	45.6	3,192	168	188	356	.217	.052	.27
2.6	9	715	45.4	3,600	177	179	356	.250	.050	.30
2.8	10	805	45.2	4,050	186	169	355	.270	.050	.32
3.0	10	900	45.0	4,500	196	159	355	.300	.045	.34
3.2	11	1,070	44.8	5,379	206	148	354	.350	.040	.39
3.4	13	1,240	44.6	6,154	217	135	352	.390	.036	.43
3.6	3	1,410	44.4	6,926	230	132	361	.415	.035	.45
3.8	7	1,580	44.2	7,735	233	125	358	.430	.035	.46
4.0	8	1,750	44.0	8,536	240	117	357	.450	.035	.48
4.2	3	1,940	43.8	9,417	248	114	361	.470	.035	.50
4.4	7	2,130	43.6	10,289	251	107	358	.490	.033	.52
4.6	8	2,320	43.4	11,094	258	99	357	.510	.030	.54
4.8	3	2,510	43.2	11,997	266	96	.....	.....	.....	.....
5.0	9	2,700	43.0	12,900	269	87	.....	.....	.....	.....
5.5	6	3,175	42.5	14,960	278	81	.....	.....	.....	.....
6.0	12	3,650	42.0	16,900	284	69	.....	.....	.....	.....
6.5	6	4,125	41.5	19,000	296	63	.....	.....	.....	.....
7.0	13	4,600	41.0	20,500	302	50	.....	.....	.....	.....
7.5	1	5,100	40.5	22,923	315	49	.....	.....	.....	.....
8.0	5	5,600	40.0	24,880	316	44	.....	.....	.....	.....
8.5	3	6,100	39.5	26,741	321	41	.....	.....	.....	.....
9.0	4	6,600	39.0	28,587	324	37	.....	.....	.....	.....
9.5	2	9,100	38.5	30,338	328	35	.....	.....	.....	.....
10.0	5	7,600	38.0	32,072	330	30	.....	.....	.....	.....
11.0	1	8,600	37.0	35,335	335	29	.....	.....	.....	.....
12.0	2	9,600	36.0	38,386	336	27	.....	.....	.....	.....
13.0	4	10,600	35.0	41,195	338	23	.....	.....	.....	.....
14.0	3	11,600	34.0	43,792	342	20	.....	.....	.....	.....
15.0	11	12,600	33.0	46,200	345	9	.....	.....	.....	.....
20.0	7	18,080	28.0	56,000	356	2	.....	.....	.....	.....
25.0	2	23,580	23.0	61,640	363	0	.....	.....	.....	.....
	365									

An inspection of the foregoing table will show to what extent any power plant which might have been installed at this place would have been affected by variations in the flow of the river during the year 1899, the number of days on which the power would have been diminished by high and low water, respectively, and the exact amount of such diminution. For instance, the power would have varied from 1,128 horsepower gross at extreme low water to 61,640 horsepower gross at extreme high water. There would have been but one day on which 1,232 horsepower gross could not have been realized, about two hundred days on which 5,000 horsepower gross could not have been realized, and two hundred and fifty days on which 10,000

horsepower gross could not have been realized. All of these figures are for continuous power, night and day. Should it be desired to concentrate flow into day service by night storage, which is practicable at Moncure, the installation for horsepower should be doubled. For instance, 10,000 horsepower gross would have been realized on one hundred days, and the power would have fallen below that amount on two hundred days.

It should be noted that the foregoing figures are based on a head of 47 feet normal, with which head little inconvenience would be experienced from high water, the maximum flood for the year being about 25 feet, and on only nine days did the water in the tailrace rise above 15 feet and on thirty days above 10 feet. But the physical conditions indicate a development of the power under two separate heads, which would be affected by flood water to a somewhat greater degree, as will be seen from an inspection of the table. Having accurate knowledge of the periods of decreased head and partial gate, it is possible to determine the capacity of the wheels for any required installation.

Should steam auxiliary be required, the table affords the data necessary to determine the exact proportion of steam power. For example, suppose it is desired to install a plant of 3,600 horsepower for night and day service, during what proportion of the year will steam power be required, and what proportion of the total power desired should be steam power? The following table shows the way of arriving at the answer. From the table on page 12 it will be seen that during one day of the year the available water power is 1,128 horsepower; so that to insure 3,600 horsepower every day in the year it will be necessary to install a steam plant of 2,472 horsepower. During twenty-seven days of the year the available water power is 1,232 horsepower, and on those days 2,268 horsepower of steam will be required, and so on, as shown in the table.

Table showing number of days and amount of steam power required during the year 1899 to develop daily 3,600 horsepower on Deep River at Moncure, North Carolina.

Number of days on which water power of the various amounts was available.	Water power available.	Steam power required.
	Horsepower.	*Horsepower.*
1	1,128	2,472
27	1,232	2,268
48	1,335	2,265
27	1,544	2,056
18	1,724	1,876
17	1,995	1,665
9	2,260	1,340
13	2,530	1,070
8	2,839	761
9	3,192	408
9	3,600	0

Summarizing, the total horsepower days (3,600×365) are 1,314,000; the total steam horsepower days are 325,743, or nearly 25 per cent, and steam power would have been required on 177 days of the year 1899.

For conciseness, the factors of wheel efficiency and partial gate are not considered in this estimate. A fair rating of the power of Deep River at this place would be 1,500 horsepower continuous, or 3,000 horsepower for day flow only, on a basis of thirty days' deficiency by reason of the dry season and thirty days' deficiency when the head would be decreased 10 per cent or more by flood water in the tailrace.

In the article referred to Mr. Hays also shows, basing his computations upon the Geological Survey records, that while in ordinary seasons, from the run-off of a drainage area of 12 square miles on Broad River we may expect 1 horsepower per foot of fall of the stream, it will require the run-off from a drainage area of 60 square miles to furnish 1 horsepower per foot of fall on Neuse River, 45 square miles on Cape Fear, Tar, Deep, and Haw rivers, 30 square miles on Roanoke River, 20 square miles on Yadkin River, 15 square miles on lower Catawba River, and 12 square miles on upper Catawba and French Broad rivers.

As the number of stations on any one stream is necessarily limited, it becomes desirable to apply the data obtained at one point to other points on the same stream. This may be done by determining the relative drainage areas. It should be noted, however, that the efficiency of a stream increases as the headwaters are approached. During the dry season of 1897 Yadkin River had nearly 33⅓ per cent greater efficiency at Salisbury than at Norwood, and the Catawba had nearly 20 per cent greater efficiency at the Catawba station, near Hickory, North Carolina, than it had at Rockhill, South Carolina. Neuse River has the lowest efficiency of any stream in North Carolina, its drainage area lying wholly in the eastern section of the State, where the geological formation is not conducive to perennial springs, and during the dry season of 1897 the run-off for that river at Selma, North Carolina, was only 15 per cent of that of Broad River at Gaffney, South Carolina, for the same drainage area. The drainage area of the latter stream lies wholly in the mountains; that of the former stream lies in the lowlands. In the year 1897 the maximum flood of the Neuse was 105 times its minimum flow, while the maximum flood of the Broad was only 24 times its minimum flow. These two streams are excellent illustrations of the effect of physical conditions on the efficiency of flow, and the physical conditions are almost invariably such as to give greater efficiency as the headwaters are approached.

A knowledge of the greatest flood volume of a stream is most essential in the construction of hydraulic works. Experience has shown that the failure of dams may in nearly every case be attributed to lack of sufficient knowledge of the floods which they will be required to withstand. In this respect also streams are very dissimilar. For

instance: The maximum flood of the Roanoke in 1897 (gage height of 28 feet at Neal, North Carolina) was about 7 second-feet per square mile. In the same year Tar River gave a flood discharge of 6 second-feet, Neuse River a flood discharge of 7 second-feet, Cape Fear River 8 second-feet, Yadkin River 10 second-feet, and Broad River 13 second-feet, while the Catawba developed a flood of 26 second-feet to the square mile. The greatest flood ever recorded on the Cape Fear at Fayetteville, North Carolina, was a gage height of 58 feet above low water and a discharge of only 13 second-feet per square mile, or half that of the Catawba in 1897.

MEASUREMENT OF SEDIMENT.[1]

The following method has been employed for measuring the amount of sediment held in suspension by streams in Arizona: By means of a small bottle attached to a hollow rod, samples were taken from various parts of the stream and collected in a bucket. After thorough mixing, the water in the bucket represented as nearly as possible the average condition of the water flowing in the stream. A measured quantity of this water, usually 100 cubic centimeters, was placed in a tubular graduate of glass divided into cubic centimeters. Ordinarily this was allowed twenty-four hours to settle, but if longer time was required for thorough settlement, it was allowed. The clear liquid was then decanted and rejected, leaving a small quantity of water with the sediment in the bottom. If the amount of sediment was inconsiderable, another sample was added, taken on the day following that on which the first sample was procured, and after settlement it was decanted in like manner, the process being repeated from day to day until a sufficient quantity of sediment had accumulated to make a reading on the scale of the glass graduate. Under ordinary conditions, when the stream was not in flood, it sometimes required thirty days to accumulate samples which would show 2 or 3 cubic centimeters of sediment, but at times of flood a large quantity of sediment was sometimes obtained from a single sample of 100 cubic centimeters, in which case the quantity of sediment was ascertained by reading, and a new determination was started on the following day. The total quantity of sediment obtained from the samples or series of samples was divided by the quantity of water used in the accumulation, and a ratio was thus established which was applied to the total volume flowing in the river.

It was found that the mud obtained from these samples was of a very thin consistency and contracted greatly upon being dried. To determine the amount of this contraction for Gila River water a number of laboratory tests were made. The residue was dried at 100° C., and the dried material was weighed, its specific gravity also being determined. As might be expected, the results of these tests were by

[1] Report of A. P. Davis.

no means uniform, but the mean of the tests made indicated about one part of dry matter to five parts of mud, which factor has been used in reducing observations of this kind. In other words, it is assumed that after complete settlement the mud in the bottom of the water sample consists of one part of solid matter and four parts of water.

The foregoing method requires very little skill and time, and the apparatus used is extremely simple. The error of the determinations lies mainly in the assumption of the factor used in reducing the mud to solid matter, and it is probably considerable.

Another method which is now undergoing test is as follows: Samples of water are obtained in the manner already described, and a measured quantity is poured upon an ordinary filter paper in a funnel and is filtered as in the chemical laboratory. If the residue is inconsiderable the process is repeated until a measurable quantity of sediment is obtained, which is dried and afterwards is weighed. The reduction is made in the same way as in the first method, that is, the quantity of water used is to the sediment obtained as the quantity of water flowing in the stream is to the quantity of silt carried in suspension.

This method, to be used with any considerable degree of accuracy, requires the use of a pair of delicate scales. The error of the determination consists partly in the errors of observations, which would be greater than by the first method, unless great skill is employed in the filtering and weighing of the sediment, but chiefly in determining or assuming the specific gravity of the silt obtained, as it is volume and not weight that ordinarily is required. It is believed, however, that where the necessary instruments and skill are available it is much more accurate than the first method.

The method of estimating the turbidity by measurement of opacity is inaccurate in any country where the water is stained to any considerable degree by leaves and roots with which it comes in contact. This is clearly shown by the writer's experience on sluggish rivers in Nicaragua, notably the San Francisco and Deseado rivers. On the latter stream it was sometimes desired to gage the river in the absence of the electric recording apparatus, but attempts to do this showed that it was impossible to observe the revolutions of the meter more than 6 inches beneath the surface of the stream, owing to the opacity of the water. At the same time sediment samples were settled and accumulated for sixty days without yielding any measurable quantity of solid matter, showing that the water was free from sediment, the dense opacity being caused entirely by vegetable stains produced by the leaves, roots, and bark of the dense tropical vegetation with which it came in contact. However, vegetable stains and sediment are seldom found in considerable degree in the same water, for only sluggish waters remain long enough in contact with vegetation to become colored, and these carry but little sediment, so that in most

cases opacity observations can profitably be used by carefully noting the true cause of the opacity.

Mr. Allen Hazen, member American Society of Civil Engineers, after reading the foregoing description, made the following comments:

Of course the aggregate amount of sediment carried by a stream, considered with reference to its tendency to fill a reservoir, is very different from the average amount of sediment carried by the water from a waterworks standpoint. In the first case, the maximum amount of sediment occurs, generally speaking, at times of maximum discharge, and, for most streams with which I am familiar, so large a percentage of the sediment as to be practically the whole of it would be carried by the water in floods occupying but a few days in each year. The average amount of sediment in the water taken for waterworks purposes, on the other hand, represents the average of the amounts in substantially equal volumes of water taken from the stream each day in the year. In comparing different streams with one another, I think it would make a great difference which method was used.

The statements regarding the colors of the waters in Nicaragua are of great interest. Of course color is an element of disturbance in any optical method of measuring turbidity. Computations and experiments conducted by Mr. Whipple and myself show, however, that water with a color which will be tolerated as a municipal supply, or which is capable of being decolorized at a reasonable cost, is not so highly colored as to affect the turbidity reading to a notable extent. The most deeply colored waters with which I have had experience have not much exceeded 2.00 on the platinum scale. The waters from Nicaragua referred to must have had colors many times deeper than this, and it would be very interesting to have accurate determinations of their colors made.

Mr. George W. Fuller, associate member of the American Society of Civil Engineers, made the following comments upon the methods described:

The determination of turbidity is a very important matter in connection with various water problems. Various methods are practiced for its determination, and thus far no standard method obtains. Apparently the "standard silica solution" is going to accomplish a great deal in this direction. Careful studies of this method are now being made at New Orleans, and the results are very promising. Messrs. Whipple and Jackson first described the standard silica solution method in the "Technology Quarterly," Vol. XII, No. 4.

The method used by Mr. Davis in Arizona was tried by myself at Louisville for several months, side by side with gravimetric determinations. It was found that the individual results were quite variable, although for the purpose for which he used them the method is probably much more satisfactory than under the conditions to which it was put in getting an accurate record of the rapidly changing Ohio River water.

The second method mentioned by Mr. Davis strikes me as a more satisfactory one, when reasonable facilities are available for carrying it out. It does not appear to me that it is so much the specific gravity of the sediment as the percentage of voids as actually found in the bottom of the reservoirs, that would materially affect the correctness of the estimates as to volume. This method has been used for a great many years on the Missouri and Mississippi rivers, and is described in the European text-books on water analysis. With coarse-grained turbidity it seems to work very well, and, all things considered, I presume that as a laboratory method it is entitled to high rating.

With the Ohio River water, which contains a considerable percentage of very

fine clay, this method was not found to be practicable, as the clay would pass through a dozen or more thicknesses of the finest filter paper, to say nothing of the length of time necessary for filtering the clay. On that work it was the practice to determine the total residue on evaporation of the water as collected, and then of the filtrate after passing the water through a Pasteur filter. The suspended matter could then be obtained by the difference, and the results were quite satisfactory when the amount of sediment was not too low. There is a slight error in this procedure, due to the Pasteur filter sometimes absorbing and sometimes giving off dissolved substances. Taking everything into consideration, the standard silica solution seems to be the more practicable procedure at present for routine work, especially when used with a diaphanometer (see Technology Quarterly, Vol. XII, No. 2). At New Orleans this method is being depended upon very largely, although the weight of suspended matter is being determined by the same method as at Cincinnati, as frequently as time allows, in order to establish the ratio between the silica turbidity and the weight of suspended matter.

There are, of course, many instances where turbidity and dissolved color are both present in the same water. As a general proposition, however, this does not seem to be true, as the very highly turbid waters below the glacial drift formation do not generally contain much color. Strictly speaking, however, such a statement would apply only to large rivers, and not to very small streams, which are influenced by swamps. On the other hand, those streams for the most part on the drift or in the neighborhood of swamps, and which are highly colored, do not, as a rule, contain sufficient turbidity to justify any elaborate records of the suspended matter.

In future years the amount of dissolved vegetable matter giving color to a water is bound to be a factor. Information upon this subject is very desirable. As suggested by Mr. Hazen, it is likely that this matter could best be handled in the field by a graduated color scale on glass, the different tints or depths of color to be obtained progressively by the use of different shades of color, or different thicknesses of glass, as found most expedient.

TESTS TO DETERMINE THE ACCURACY OF DISCHARGE MEASUREMENTS OF NEW YORK STATE CANALS AND FEEDERS.[1]

During the summer of 1900 the State of New York undertook investigations to determine the flow, seepage, and evaporation of its canals. The work was under the immediate supervision of Mr. E. Kuichling, member American Society of Civil Engineers. Careful measurements were made by means of floats and current meters at various points on the Erie Canal and its branches. After completing the work it was thought wise to investigate the methods and determine the accuracy of the measurements taken. These special experiments were made under the direction of Prof. G. S. Williams, in charge of the hydraulic laboratory at Cornell University, assisted by the writer, by Mr. W. P. Boright, civil engineer, and by members of the senior class in experimental hydraulics at the university. Float and meter measurements were made in the flume at the hydraulic laboratory, and the results were compared to determine the variation between the two methods of measurement. It is thought that a brief description of the general method applied and of the results obtained will be of interest to engineers.

[1] Report of E. C. Murphy.

The movement of water in canals and feeders is quite different from that of a natural stream. This difference is due mainly to lockage and to irregular feed from one section to another, which cause rapid fluctuations of the surface level and of the velocity—in some cases the surface level dropped several inches in a few seconds and the velocity increased from 25 to 50 per cent. The disturbance of the water at a given place may be due either to lockage or to feed at one or both ends of the section. The water that is needed in a lower level should be passed through a culvert. Ordinarily, however, it is passed through the lock by opening one or more valves in the gates. In some cases four such valves are opened at once for a few minutes, but as a rule only one is opened for a longer time. It will readily be seen, then, that when a boat is passing in or out of a section which is being gaged, or is entering or leaving short adjacent sections, the water in that section is in a very disturbed condition. The length of time required for it to return to a normal condition after one of these disturbances, depends to some extent on the velocity of the water. The greater the velocity, the quicker it will return to normal condition; for very low velocities it requires hours to quiet down. In one case, with a steady gage, it was impossible to obtain a single gaging (which required only five minutes) in a whole day. It will readily be seen that, to be of any value, discharge measurements under such conditions must be made quickly. Even when the surface did not fluctuate the work was often stopped by one or more boats passing the place of measurement. Instantaneous measurements of discharge and a continuous record of surface fluctuations at the point of measurement are desirable. This was approximated by making a rod gaging in from four to ten minutes and reading the surface fluctuations every thirty to sixty seconds.

Two instruments were used to measure the velocity, viz, float rods and current meter. The float rods used were 1.90 inches in diameter, of wood, and weighted at the lower end with iron pipe and lead of the same diameter, so as to float upright. By adjusting the weight at the lower end the immersed length of the rod may bear any ratio to the depth of the water in which it floats. If the immersed length is equal to the depth, the speed or velocity of the rod is very nearly that of the average velocity of the water in the vertical plane in which the rod moves. On account of the action of the wind on the part of the rod which projected above the surface of the water, that part of the rod was limited in length to about 8 inches. Extensions of the rod were used when the depth required them.

In this work it was not possible to have the depth of immersion of the rods more than about 90 per cent of the depth of the water, on account of inequalities in the bottoms and sides of the canals. Neither was it advisable to have rods of many lengths, on account of the labor required to transport them from place to place. Rods of three lengths were carried, viz, 18, 40, and 80 inches, which, with a set of exten-

while the float rods give the mean velocity in a vertical at one observation. The meter will give the velocity at any point in the depth; the rods will give the mean velocity only. The meter can also be used where the rods can not, and it is a less expensive method of measuring discharge.

The laboratory tests cover a range of velocities of 0.25, 0.50, 1, and 1.50 feet per second for channel depths of 7.5, 8.3, and 9 feet, and 0.5, 1, 1.50, and 2 feet per second for a channel depth of 6 feet. Two depths of immersion were used with each velocity, viz, 75 per cent and 90 per cent. Every velocity, depth of channel, and depth of immersion found in the field work could not be duplicated in the laboratory tests, so that the foregoing range covers only the most important cases. The laboratory experiments were made in the same way as the field experiments, using the same lengths of run and the same distance from the point of starting the floats to the upper chain.

The conditions of the flow were not quite the same in the field tests as in the laboratory tests. The Lockport comparison was made in a rock cut, with nearly vertical rough rock sides and a rough bottom. The meter section was 62.5 feet wide, 8.5 to 9 feet deep, and about 700 feet above the point where the rod measurements were made. The section where the rod discharge was measured was 65 feet wide and 8.5 to 10 feet deep. There was undisturbed flow for at least a half mile on each side of the place of measurement. The Rochester comparisons were made on the Rochester Aqueduct, which has very smooth and nearly vertical sides. The width was 43 feet, the depth 7.5 feet. The flow was undisturbed for a half mile above the point of measurement, but there was a very sharp curve less than 100 feet below the place where the rods were used. The Boonville comparisons were made in an earth cut with side slopes varying so as to make a gradual transition from the flat bottom to the top. The surface width was 43 feet, the greatest depth 7.7 feet. There was unobstructed flow for a quarter of a mile above the point of measurement and for a half mile below it. The sections where the Glens Falls comparisons were made have nearly vertical sides. The width is 32 feet, the greatest depth 6 feet. There is a bend in the feeder about 700 feet above the upper section, and another bend about 1,000 feet below the lower section. The meter section was between the rod-measuring sections, 75 to 100 feet distant from them.

The Cornell University canal has vertical sides of concrete and a concrete bottom. One side is at present roughened somewhat on account of the lining being cut out in places preparatory to its repair. The canal is 16 feet wide, 10 feet deep, and 415 feet long, with a standard sharp-crested weir, and with gates at the upper end for admitting water and at the lower end for controlling the depth of water in the canal, and with piezometers and gages for measuring accurately the head on the weir and the depth of water in the canal.

The discharge of the weir is known for any head on it, so that a simple measurement of the head gives the discharge of the canal. The discharge can then be measured with rods or current meter and compared with the weir discharge, and thus the error by the rods or meter be determined. The weir is 220 feet above the point where the rod measurements were made, and 15 feet below it are baffleboards for quieting the water. About 70 feet below the meter section is a bulkhead, and at the side of it are two rectangular gates. During the experiments the water passed out of the canal through two horizontal slits in the bulkhead, used to secure velocity in all of the water section. At the higher velocities some water passed out under one of these gates. The width of the Cornell channel was only from one-fourth to one-half that of the channels in which the field comparisons were made. At Cornell the water was probably moving in a more disturbed condition than in the canals and feeders. The retarding effect of bottom and sides was less in the former channel than in the latter, and the depth of water and the area of the channel were measured with greater accuracy at Cornell than on the canals.

The results of the field comparisons are given in the following table:

Results of field comparisons.

Place.	Date.	V'r.	V''r.	Vm.	Q'r.	Q''r.	Qm.	Q'r—Q''r in Q'r.	Q'r—Qm in Q'r.	Q''r—Qm in Q''r.
	1900.	*Feet.*	*Feet.*	*Feet.*	*Cu. ft. per sec.*	*Cu. ft. per sec.*	*Cu. ft. per sec.*	*Per ct.*	*Per ct.*	*Per ct.*
Lockport	Sept. 14	.....	1.440	1.531	.....	805.70	799.85	.....	.....	+0.73
Rochester	Sept. 20	.....	.704	.736	.....	235.12	251.57	.....	.....	—7.00
Do	Sept. 21	.893	.884	.....	297.82	302.04	.....	—1.41	.....	.....
Do	....do...	.....	.889	.929	.....	301.35	310.83	.....	.....	—3.14
Do	....do...	.....	.899	.939	.....	307.08	317.54	.....	.....	—3.40
Boonville	Sept. 25	1.073	.813	.....	227.22	227.51	.....	— .13	.....	.....
Do	....do...	.....	.850	1.079	.....	238.81	228.66	.....	.....	+4.25
Do	....do...	.....	.836	1.111	.....	233.54	234.70	.....	.....	— .49
Glens Falls	Oct. 8	.821	.690	.....	164.73	172.77	161.50	—4.88	+1.94	+6.52
Do	....do...	.756	.640	.....	149.66	158.57	155.23	—5.92	—3.72	+2.11

V'r = mean velocity found by party No. 1, with rods.
V''r = mean velocity found by party No. 2, with rods.
Vm = mean velocity found with meter, using six-tenths method.
Q'r = discharge found by party No. 1, with rods.
Q''r = discharge found by party No. 2, with rods.
Qm = discharge found with meter, using six-tenths method.

In the Lockport and Glens Falls comparisons the meter was suspended from a bridge by means of an insulated wire. In the Rochester and Boonville comparisons it was suspended from a boat by means of the same wire. The conditions were very good during the Lockport comparisons. There was very little wind, the surface did not fluctuate, there was a good measurable velocity in all parts of the cross sections, and no curve near to disturb the direct flow of the water. The meter discharge will be seen to agree closely with the rod discharge.

The conditions were not so good in the Rochester comparisons. The lower gaging station was too near a curve in the canal. The time of run of the individual rods at the same point differs by a considerable amount, much more than at the upper station. On the afternoon of September 20 the surface did not fluctuate much, but rain interfered somewhat with the work. The meter discharge for that day is 7 per cent greater than the corresponding rod discharge. The next day, September 21, the surface fluctuated somewhat, but not more than while much of the regular gaging work was being done. The rod discharge measurements for that day differ by 1.41 per cent, and the average meter discharge is larger than the rod discharge by 3.3 per cent.

At Boonville the conditions were all good. The rod discharges agree closely, as does one of the meter measurements with the corresponding rod measurement.

The Glens Falls comparisons are not very good—the upper gaging station was too near a curve, and there was a strong probability of leakage from the feeder between the points where the rod measurements were made. There was no leakage visible on the surface, but the point where the measurements were made is only a short distance below the portion of the feeder which is noted for its large leakage, so that there probably was some leakage here, although it does not show on the surface.

For the reasons stated we believe that the results of the Glens Falls comparisons should be given little weight, and that those at Rochester should be given only half the weight of those at Lockport and Boonville.

Results of experiments at Cornell hydraulic laboratory.[1]

Date.	Rod gagings in canal.					
	Referred to standard weir.	Depth of water.	Immersion, 75 per cent.		Immersion, 90 per cent.	
			Velocity.	Discharge.	Velocity.	Discharge.
1900.	*Cu. ft. per sec.*	*Feet.*	*Ft. per sec.*	*Cu. ft. per sec.*	*Ft. per sec.*	*Cu. ft. per sec.*
October 24	214.6	9.36	1.42	213.8	----------	----------
Do	140.5	9.25	.96	143.0	0.95	141.0
Do	71.7	9.34	.49	73.5	.48	71.4
Do	38.2	8.81	.28	39.1	.27	38.6
October 26	178.6	7.54	1.51	183.1	1.53	184.8
Do	125.3	7.55	1.08	131.0	1.06	128.3
October 27	61.3	7.47	.53	63.7	.52	62.5
Do	30.9	7.38	.27	31.6	.26	30.3
Do	196.4	6.16	2.07	204.7	2.04	202.4
Do	132.4	6.30	1.35	136.7	1.34	135.8
Do	91.1	6.23	.95	95.4	.94	94.5
Do	50.3	5.85	.56	52.2	.55	51.6
October 31	198.3	8.37	1.54	204.9	1.51	202.4
Do	125.3	8.39	.96	129.1	----------	----------
October 29	65.4	8.37	.51	68.8	.50	67.3
November 1	31.1	8.26	.25	33.1	.24	32.1

[1] A recomputation of this data gave results slightly different from those contained in this report.

Results of experiments at Cornell hydraulic laboratory—Continued.

Date.	Referred to standard weir.	Depth of water.	Current-meter gagings in canal.							
			Small Price meter (No. 363).				Small Price meter (No. 351), ordinary method.			
			Six-tenths method.		Integrating method.		Analytical.		Graphical.	
			Velocity.	Discharge.	Velocity.	Discharge.	Velocity.	Discharge.	Velocity.	Discharge.
1900.	*Cu. ft. per sec.*	*Feet.*	*Ft. per sec.*	*Cu. ft. per sec.*	*Ft. per sec.*	*Cu. ft. per sec.*	*Ft. per sec.*	*Cu. ft. per sec.*	*Ft. per sec.*	*Cu. ft. per sec.*
October 24	216.1	9.46	1.47	223.0	1.43	217.5	1.45	220.4	1.44	219.0
Do	140.5	9.37	.99	148.5	.98	148.1	.95	143.1	.95	143.2
Do	71.7	9.34	.49	73.2	.48	72.7	.48	71.5	.48	72.1
Do	38.3	8.92	.28	40.1	.31	44.4	.24	33.3	.25	35.5
October 26	178.6	7.67	1.49	184.1	1.52	187.8	1.41	174.7	1.43	176.2
Do	125.3	7.64	1.05	128.9	1.08	131.7	.98	120.3	1.00	123.1
October 27	61.3	7.58	.52	63.8	.52	63.8	.50	61.3	.50	60.7
Do	31.2	7.56	.26	31.5	.31	38.1	.23	28.1	.24	29.2
Do	196.4	6.28	1.99	201.4	2.03	205.4	1.86	188.2	1.90	192.1
Do	132.4	6.40	1.32	136.0	1.34	138.1	1.20	123.8	1.24	127.3
Do	91.1	6.32	.94	95.7	.92	93.8	.82	83.9	.85	87.3
Do	50.3	6.01	.54	52.7	.56	54.2	.48	46.5	.49	48.0
October 31	198.3	8.47	1.51	206.5	1.55	211.7	1.38	187.8	1.44	195.8
Do	125.3	8.51	.95	130.0	1.00	137.3	.84	114.5	.86	117.3
October 29	65.3	8.48	.50	67.7	.49	67.1	.44	60.5	.45	61.0
November 1	31.2	8.46	.24	32.7	.32	43.4	.19	25.2	.21	28.9

Date.	True velocity at meter station.	Error in discharge computed by different methods when referred to standard weir.						
		Rod measurements.		Meter measurements.				
				Six-tenths method.				
		Immersion, 75 per cent.	Immersion, 90 per cent.	Analytical.	Graphical.	Integrating method.	Ordinary method.	Ordinary method computed from velocity curves.
1900.	*Ft. per sec.*	*Per cent.*	*Per cent.*	*Per cent.*	*Per cent.*	*Per cent.*	*Per cent.*	*Per cent.*
October 24	1.41	+3.64	.....	−4.11	−3.17	− 0.63	− 1.97	−1.13
Do	.93	−1.74	−0.37	−6.13	−5.70	− 5.37	− 1.84	−1.94
Do	.48	−2.56	+ .28	−2.08	−2.08	− 1.59	+ .24	− .64
Do	.27	−2.44	−1.02	−6.00	−4.86	−16.16	+12.84	+7.02
October 26	1.44	−2.57	−3.51	−3.09	−3.09	− 5.17	+ 2.17	+1.34
Do	1.02	−4.55	−2.40	−2.76	−2.86	− 5.12	+ 3.60	−1.73
October 27	.50	−3.94	−1.93	−4.88	−4.08	− 4.08	− .08	+ .90
Do	.26	−2.24	+2.01	−1.04	−1.04	−22.10	+10.00	+6.44
Do	1.94	−4.23	−3.06	−1.46	−2.55	− 4.86	+ 4.14	+2.20
Do	1.28	−3.24	−2.52	−3.18	−2.71	− 4.27	+ 6.49	+3.84
Do	.89	−4.75	−3.74	−5.32	−5.08	− 2.96	+ 7.89	+4.06
Do	.52	−3.74	−2.62	−5.92	−4.75	− 7.65	+ 7.67	+5.72
October 31	1.45	−3.36	−2.11	−3.71	−4.15	− 6.76	+ 5.28	+1.22
Do	.92	−3.05	.....	−3.75	−3.75	− 9.54	+ 8.60	+6.41
October 29	.48	−5.14	−2.88	− .81	−2.35	− 2.80	+ 7.44	+6.62
November 1	.23	−6.14	−3.16	−2.47	−4.65	−39.30	+19.34	+7.57

NOTE.—Minus sign indicates that the discharge is greater than that by standard weir; plus sign indicates that it is less than that by standard weir.

Referring to the Rochester comparisons, it must not be inferred that because the rod discharges found by the two parties on September 21 agree closely, and the meter and rod discharges of the 20th differ by 7 per cent of the rod discharge, that this difference is

all due to the meter. At least half of it is chargeable to the rod measurement. The two sets of velocity readings given by the meter on the 20th agree closely; there is nothing about them that will account for this large difference. The time of run of the rods, however, differs by a considerable amount, as already stated. The rod-measuring party was hindered by the rain and by the refusal of one of the men to work in the rain.

The results of the experiment work are given in the foregoing tables. The method of procedure was to open the head gates until the head on the weir was such as to give the desired discharge; then the slits in the bulkhead and the height of the gates at the lower end of the canal were adjusted so as to give the desired depth of water in the canal. As soon as the flow of water became steady, three parties began measuring the discharge of the canal, one with rods and two with current meters. Two lots of rods were used, one lot of 75 per cent depth of immersion, the other lot of 90 per cent depth of immersion; the former rods were lettered, the latter rods were numbered, so that they could easily be recognized as they passed under each wire. One of the current-meter parties measured the discharge in the ordinary way; the other party used the six-tenths and the integrating methods, and after completing their measurements they assisted the rod-measuring party, four stop watches being then employed and the number of rods observed in a given time being nearly doubled. The rod work was continued until the other meter party completed its measurement. During the experiment a gage giving the head on the weir and two gages giving the depth of water in the canal and the slope of the water surface, respectively, were read every 30 seconds. As a check on the weir gages and canal gage readings other readings were taken with a hook gage several times during the experiment. The length of run of the rods, or the distance between the chains was as follows: 25 feet for the 2-foot velocity, with a starting distance of 10 feet; 20 feet for the 1-foot and the 1.5-foot velocities, with a 10-foot starting run; 10 feet for the 0.5-foot velocity, with a 10-foot starting run; and 7 feet for the 0.25-foot velocity, with an 8-foot starting run. The number of rod observations made in an experiment varied from 50 to 150. The observations of each rod-discharge experiment were platted, using the distance from the side of the canal as abscissa and the time of run as ordinate, and a mean time curve was drawn. The mean time was found from this curve, and from that the mean velocity, which multiplied by the cross-sectional area gave the discharge.

In the six-tenths method the meter was held with its center at six-tenths of the depth from the surface, and, successively, at 1, 3, 7, 9, 13, and 15 feet from one side of the canal, and the revolutions in two consecutive periods of 50 seconds each were counted from the indications of an electric buzzer. The meter was then held at the same distances from the other side of the canal, and the revolutions in two

consecutive 50-second periods were counted as before. These observations gave four measurements of velocity, covering a period of 50 seconds each, in these six verticals, at six-tenths of the depth below the surface. These points were platted, using the distance from the side of the canal as abscissa and the revolutions in 100 seconds as ordinate, and a mean curve was drawn. The mean number of revolutions for the whole cross section was computed from this curve and converted into mean velocity by the use of a rating table. The discharge was then found by multiplying the mean velocity by the cross-sectional area. The discharge was also found, analytically, without platting the points and drawing a mean curve. The difference between the weir discharge and the meter discharge, expressed in per cent, as found by these two processes, is given in the table on page 24. As a rule, the discharge found by using the curve is more accurate than that found without it.

In the integrating method the meter was moved in the following way: Its center started at 0.5 foot from the bottom and 1 foot from the side of the canal, and passed slowly to the surface at 4.5 feet from the side, then to 0.5 foot from the bottom and 8 feet from the side; then to the surface at 11.5 feet from the side, then to 0.5 foot from the bottom and 15 feet from the side. The time at the beginning and end of this operation was noted, and the revolutions of the meter wheel were counted, as indicated by the buzzer. The meter was then carried back over the same path at about the same speed, the time of passage being noted and the revolutions of the meter wheel counted as before. The discharge was computed by dividing the sum of the revolutions of the meter wheel in the passage across the canal and back again by the corresponding time, converting this into velocity and multiplying by the cross-sectional area.

In the ordinary method the meter was held by a rod at five points in the same six verticals in which the other meter was held, and the number of revolutions of the meter wheel, in single periods of 60 seconds each, were read from a recorder. The points in the verticals were usually 1¾ inches, 1 foot, and 2 feet from the bottom, the surface, and at one other point between the two. The recorder indicated only complete revolutions, while with the buzzer quarters of a revolution could be recognized. The mean velocity shown by the observations with this meter has been computed in two ways: (1) The mean number of revolutions per second of the meter wheel while being moved from one side of the canal to the other, at the five depths of immersion, have been computed and platted and a curve drawn through them, from which the mean number of revolutions per second for the whole cross section has been found, and this converted into mean velocity by the use of the rating table; and (2) the mean velocity has been computed by using with these observations the observations obtained at the same time with the other meter, using the six-tenths method, the mean revolutions per second at the different depths being converted

into velocity and platted on a large scale, the velocity at six-tenths the depth being platted therewith and a mean velocity curve drawn among the points (not necessarily connecting them), from which the mean velocity for the whole cross section was found. The discharge found from the mean velocity computed in the latter way is given in the table on page 24, also the velocity found by the ordinary method.

Meter No. 351 used in the ordinary way did not work well on November 1. The wheel did not turn continuously for the small velocity 0.229 foot per second, but would stop for a few seconds and then start again. The other meter (No. 363) worked much better for the same velocity. The rating tables, however, indicate that the latter meter will measure a smaller velocity than the former. It should be said that the rating table for meter No. 351, which was used to convert revolutions into velocity, is for the meter suspended by a cable and not held by a rod. There was not sufficient time to rate it on a rod before making the computations.

In the table on pages 23 and 24 two values of depths are given; the one in the third column on page 23 is the mean depth for the 40 feet of distance passed over by the rods, the other (third column, top of page 24) is the mean depth at the meter station, and is for a distance of 15 feet. Two values of discharge of the standard weir are given for each experiment, the one in the second column being the average for the period during which discharge was measured by rods and meter, using the ordinary method; the other being for the period during which the discharge was measured by meter, using the six-tenths and integrating methods.

It will be seen from the table on pages 23 and 24 that the rod discharge and the meter discharge for the six-tenths method and for the integrating method are greater than the weir discharge, and that the meter discharge by the ordinary method is less than the weir discharge. The sign used is that of the correction to be applied to the meter or rod discharge to obtain the standard weir discharge. The rod discharge for 90-per cent rod immersion is on an average about 2 per cent greater than the weir discharge, and the rod discharge for 75-per cent rod immersion and the meter discharge by the six-tenths method are each on an average 3.5 per cent greater than the weir discharge. The variation from the weir discharge, which we will call the error, is in each case slightly greater for the lower velocities than for the higher velocities. The extreme percentage difference between any discharge and the mean of all the corresponding discharges is about 2.6 per cent. The error in the discharge as found with the current meter, using the ordinary method, is seen to increase as the velocity decreases, and the error in the meter discharge as found by the integrating method is seen to also increase as the velocity decreases. It also increases as the speed at which the meter is carried through the water increases. This we know must be the case, since the velocity indicated by the meter is the resultant of the velocity of the water

and the velocity of the meter as it is moved through the water. When the velocity of the water is small, doubling the speed of the meter will nearly double the indicated velocity. In the following table is given the speed, in feet per second, of the meter, also the percentage of difference between the true velocity and the velocity as found by the integrating method. For a given speed of meter, the error is seen to be greater for the low velocities than for the higher, and for any given velocity of water the error increases with the speed of the meter.

Table showing effect of change of speed of meter in the integrating method, computed from experiments at Cornell University.

Date.	Depth of water.	(S) Speed of meter, integrating method.	(V) True velocity.	(V_1) Velocity by integrating method.	$\frac{V-V_1}{V}$.
1900.	*Feet.*	*Ft. per sec.*	*Ft. per sec.*	*Ft. per sec.*	*Per cent.*
October 24	9.460	0.118	1.409	1.429	— 1.42
Do	9.374	.142	.932	.982	— 5.36
Do	9.338	.164	.477	.485	— 1.68
Do	8.918	.107	.265	.308	—16.23
October 26	7.673	.137	1.444	1.519	— 5.20
Do	7.640	.135	1.018	1.079	— 5.99
October 27	7.582	.143	.502	.522	— 3.99
Do	7.557	.128	.255	.312	—22.35
Do	6.278	.180	1.940	2.030	— 4.64
Do	6.399	.154	1.283	1.338	— 4.29
Do	6.319	.188	.893	.920	— 3.02
Do	6.005	.142	.515	.555	— 7.77
October 29	8.476	.151	.478	.492	—29.29
October 31	8.470	.293	1.454	1.552	— 6.74
Do	8.508	.258	.915	1.002	— 9.51
November 1	8.455	.185	.229	.319	—39.30
November 29	8.476	.301	.478	.572	—19.67

CONCLUSIONS.

Assuming that the discharge given for the Cornell University weir is correct, the greatest percentage error in discharge when measured with rods of 90 per cent depth of immersion is 3.74 per cent. The range of percentage errors, omitting that on November 27 for velocity, 0.255 foot per second, is 4 per cent, the mean error for all depths and velocities is 2 per cent, and the greatest variation from this mean is 2.5 per cent. The greatest percentage error in the discharge when measured with rods of 75 per cent depth of immersion is 6.14 per cent. The range of these percentage errors, omitting that on November 24 for the velocity of 1.409 feet per second, is 4.4 per cent. The mean error for all depths and velocities is 3.5 per cent, and the greatest departure from this mean is 2.6 per cent. The greatest percentage error in the discharge as found with the current meter, using the six-tenths method, is 5.7 per cent, and the range of these percentage errors is 4.7 per cent. The mean percentage error for all depths and velocities is 3.5 per cent, and the greatest departure from this mean is 2.6 per cent.

From these facts we may conclude (1) that a discharge measure-

ment of a canal or feeder may be in error 6 per cent when its mean velocity is small, but that, as a rule, the error will not be more than 3.5 per cent, and (2) that a measurement of loss from a section, which is mainly the difference between two simultaneous discharge measurements, may be in error 4.7 per cent—the largest difference that can be obtained by subtracting any two of the percentage errors in the third, fourth, and sixth columns of the table on page 24, except the two just mentioned, and it is very unlikely that this worst combination of errors will ever occur; but it is quite likely that the errors made by the two parties will be about the same in magnitude and have the same sign, in which case the measured percolation is nearly free from error. The average error in the measured percolation is probably not more than 2 per cent, and does not depend on the correctness of the standard weir discharge.

Of no little interest and importance is the fact, brought out by these comparisons and accuracy tests, that the current meter will give quickly and with a high degree of accuracy the discharge of canals and feeders or other channels of rectangular and trapezoidal cross section with fairly smooth bottoms.

The general subject of the relative accuracies of the various methods of measuring stream flow should be more thoroughly investigated, and it is hoped that more light will be thrown upon obscure points in river hydraulics and some improvements be suggested in the methods of measuring the flow of streams.

MEASUREMENTS AT RIVER STATIONS.

KENNEBEC RIVER AT WATERVILLE, MAINE.

This river was described by Prof. Dwight Porter in the Nineteenth Annual Report, Part IV, pages 65 to 84. Tables of its daily discharge from 1892 to 1898 will be found in Water-Supply Paper No. 27, pages 11 to 14; and similar data for 1898 and 1899 will be found in Water-Supply Paper No. 35, pages 25 and 26. The figures for the latter year, however (April to December, inclusive), were not computed with the same degree of accuracy as were those for former years, and since the publication of Water-Supply Paper No. 35 they have been recomputed, and the revised figures for the year 1899 and those for the year 1900 are presented herewith.

The computations for this station include the figuring of the discharge over the dam, through the waste gates and waste weirs, and through the various wheels of the Hollingsworth and Whitney mills. The Sunday flow at times, especially at low stages of the river, is found to be irregular, as the mill at Waterville is sometimes operated Saturday nights, drawing the water from the pond, so that on Sunday there is no flow except that due to leakage. During the dry season there is usually less water passing on Sundays than on week days, for at many of the mills on the river the dams are closed in order to allow the ponds to fill.

Daily discharge, in second-feet, of Kennebec River at Waterville, Maine, for 1899.

[Drainage area, 4,410 square miles.]

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	2,529	2,458	3,382	6,411	41,756	13,044	6,115	4,809	1,623	.....	1,390	1,456
2	2,202	2,198	3,047	6,214	39,665	12,790	5,024	3,594	2,010	1,443	2,819	1,441
3	2,274	2,187	3,353	6,160	36,589	10,647	5,363	4,442	567	1,665	4,093	747
4	2,159	2,163	3,324	6,017	36,942	11,673	3,178	4,199	1,612	1,415	4,109	2,913
5	2,452	1,580	1,960	6,207	30,922	12,264	5,258	4,498	2,070	1,969	2,995	3,567
6	2,451	2,505	3,331	6,227	30,531	11,123	5,120	4,415	2,082	1,449	3,327	3,196
7	2,449	2,173	3,563	6,433	30,125	10,563	5,076	5,064	2,070	1,476	3,106	2,015
8	1,739	2,071	3,516	7,007	29,447	12,748	5,367	5,195	.....	.....	2,637	2,070
9	2,757	2,057	3,502	7,541	26,888	12,517	3,936	4,388	2,352	1,215	2,338	1,737
10	2,513	2,152	3,428	9,001	24,803	12,506	5,142	4,128	246	940	2,119	.....
11	2,400	2,141	3,357	9,650	22,132	10,738	5,123	4,101	2,635	774	1,698	1,398
12	2,386	2,244	2,302	9,405	20,229	10,278	4,866	3,954	2,067	1,784	.....	1,160
13	2,404	2,348	3,353	11,181	18,697	6,039	5,100	2,699	2,340	1,968	1,488	2,022
14	2,436	1,964	3,423	19,700	19,643	7,030	5,950	3,964	2,370	1,391	1,970	5,223
15	1,678	1,844	3,624	23,106	14,886	6,739	5,078	3,538	2,257	492	2,017	4,410
16	2,731	2,392	3,773	38,826	15,099	7,557	4,043	3,213	1,261	1,378	2,053	4,621
17	2,439	2,434	3,501	30,532	17,307	5,913	4,866	3,007	1,298	1,667	1,823	1,861
18	2,631	2,727	3,253	30,312	15,073	6,807	5,232	3,321	1,416	1,366	1,739	2,695
19	2,434	2,198	1,960	37,096	15,071	7,290	4,948	2,643	2,005	1,096	.....	2,027
20	2,629	2,915	3,320	41,565	15,684	6,483	4,880	1,242	2,362	1,377	1,718	2,611
21	2,440	2,734	3,080	34,540	16,262	7,101	5,499	2,748	2,006	1,120	2,317	1,878
22	2,094	2,744	2,778	30,286	18,287	7,317	6,367	3,377	2,351	.....	1,748	2,340
23	2,709	2,451	2,711	39,550	17,271	7,319	5,688	2,956	2,348	1,408	2,085	2,617
24	2,154	2,745	2,837	45,724	15,900	7,516	6,509	2,534	1,073	1,334	2,035	2,068
25	2,435	2,929	2,770	41,739	15,925	5,673	5,837	2,180	2,031	1,104	2,028	.....
26	2,450	1,420	2,030	43,139	11,480	7,301	5,916	2,158	1,761	995	.....	.....
27	2,454	3,039	2,786	45,795	12,567	7,315	5,502	1,089	1,466	1,149	1,413	2,323
28	2,434	3,362	2,740	39,943	12,303	6,880	5,649	2,288	2,046	1,121	1,743	2,330
29	1,245	.....	2,998	41,584	13,064	7,038	4,170	2,203	1,951	406	1,744	1,821
30	2,513	.....	3,962	39,300	13,035	6,445	2,500	2,065	2,091	1,104	.....	1,474
31	2,453	.....	6,820	.....	12,823	.....	4,087	2,354	.....	1,086	.....	.....
Mean	2,357	2,363	3,218	24,006	21,306	8,821	5,077	3,302	1,854	1,274	2,252	2,741

Daily discharge, in second-feet, of Kennebec River at Waterville, Maine, for 1900.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	1,463	7,117	15,468	8,792	22,830	12,115	4,849	4,910	4,238	2,859	2,307	6,540
2	1,565	3,696	13,353	8,381	27,905	12,782	8,232	5,149	1,635	3,337	2,357	4,394
3	1,604	3,628	9,656	9,338	28,823	15,730	7,520	5,525	3,191	3,134	2,293	6,146
4	1,653	.....	11,445	14,047	52,268	25,091	2,763	5,601	3,180	2,876	1,502	5,968
5	1,193	3,166	6,544	15,927	48,843	19,591	6,715	4,431	2,905	2,565	1,660	4,640
6	1,452	3,159	6,240	15,544	39,585	16,103	6,255	5,357	2,913	2,168	2,260	4,583
7	1,304	3,188	9,323	19,353	34,551	14,088	6,271	4,708	2,865	859	1,604	4,508
8	2,324	3,303	4,913	27,072	28,889	12,792	4,799	4,597	2,908	3,457	1,620	4,701
9	2,060	2,925	4,947	24,219	23,561	10,930	6,465	4,573	2,276	3,715	3,819	3,548
10	1,998	3,114	4,927	19,765	28,628	10,495	6,501	4,758	2,330	3,715	17,580	4,498
11	1,751	1,615	4,373	17,053	25,277	10,083	5,767	4,462	2,871	4,051	11,521	2,978
12	1,711	2,933	4,950	20,560	22,411	10,148	4,984	2,627	2,865	4,106	13,301	3,504
13	1,009	12,148	4,932	25,463	20,804	9,661	12,281	4,651	2,500	4,003	10,314	3,851
14	696	15,696	4,799	27,373	18,378	10,168	9,601	4,716	2,851	.....	8,564	4,078
15	1,712	21,761	5,051	26,429	22,307	7,707	6,262	4,450	2,826	2,590	5,839	4,101
16	1,996	19,766	10,516	31,224	29,793	9,141	7,018	4,750	2,426	3,998	4,877	3,080
17	1,755	15,073	7,479	32,034	29,945	7,256	5,112	4,834	3,130	3,993	3,706	3,958
18	1,748	11,998	12,670	35,155	31,845	7,808	6,017	4,624	3,147	4,190	6,533	4,068
19	2,064	5,871	20,538	42,557	40,265	7,308	5,652	3,075	2,776	3,989	5,079	3,545
20	1,792	7,090	12,562	60,541	48,460	5,866	6,057	4,547	2,602	3,720	5,215	3,819
21	656	6,978	12,448	62,291	44,162	6,402	5,678	4,072	3,044	2,392	8,107	3,759
22	2,060	6,123	12,412	55,863	35,060	6,651	4,386	3,203	2,776	3,164	6,360	4,109
23	4,045	5,308	11,361	51,180	25,403	6,613	5,253	4,029	.....	3,185	12,471	1,865
24	3,748	4,117	10,742	44,064	23,333	5,015	4,923	3,649	3,137	2,621	8,620	3,896
25	3,697	18,833	9,792	38,913	21,682	6,669	4,737	.....	2,887	2,902	7,886	3,198
26	3,684	13,387	9,460	33,353	18,907	5,066	5,403	1,016	2,563	2,624	7,000	4,311
27	3,609	23,971	8,925	27,800	17,467	5,803	4,857	3,169	2,575	2,373	7,330	4,498
28	3,007	18,375	8,604	23,905	15,469	7,679	4,872	3,501	2,568	1,520	9,698	4,299
29	6,851	.....	8,391	21,793	19,171	7,189	2,344	3,629	3,169	2,902	5,214	4,035
30	4,311	.....	8,410	14,688	16,301	9,043	4,231	3,147	2,197	2,623	6,660	2,498
31	5,399	.....	8,498	.....	14,113	.....	3,725	3,454	.....	2,338	.....	4,059
Mean	2,384	9,050	9,153	28,473	28,272	10,033	5,791	4,173	2,807	3,065	6,376	4,096

COBBOSSEECONTEE RIVER AT RESERVOIR DAM NEAR AUGUSTA, MAINE.

This river is described in the Nineteenth Annual Report, Part IV, pages 79 and 80. The record of the discharge measurements from 1890 to 1899, inclusive, will be found in Water-Supply Paper No. 35, pages 28 to 33. The following measurements of daily discharge for 1900 were furnished by Mr. Alexander H. Twombly, engineer of the Forest Paper Company, of Yarmouthville, Maine. The measurements are made at the reservoir dam, where the discharge of the river is controlled by gates, except during flood stages. The gates are closed on Sundays and no water is allowed to pass. The great Cobbosseecontee dam furnishes a supplementary supply to the reservoir below, and water is occasionally drawn from it.

Daily discharge, in second-feet, of Cobbosseecontee River at upper dam near Augusta, Maine, for 1900.

[Drainage area, 230 square miles.]

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	130	220	815	776	300	300	(a)	260	260	230	180	220
2	130	220	1,453	495	300	300	280	260	(a)	220	165	(a)
3	130	220	2,316	417	300	(a)	280	260	260	190	140	220
4	125	(a)	2,055	656	413	300	(a)	260	260	175	(a)	220
5	125	220	1,911	977	803	280	280	(a)	240	175	130	220
6	125	220	1,622	907	934	280	280	260	240	150	130	220
7	(a)	220	1,481	1,087	831	280	280	260	240	(a)	130	220
8	130	220	1,295	1,379	606	280	(a)	260	240	160	130	220
9	110	220	1,155	1,380	456	280	280	260	(a)	175	200	(a)
10	100	220	1,155	1,342	413	(a)	280	260	240	180	220	220
11	90	(a)	1,153	1,297	348	280	280	260	240	190	(a)	220
12	90	220	1,116	1,297	300	280	280	(a)	240	220	220	240
13	90	220	1,037	1,297	(a)	280	280	260	235	220	220	240
14	(a)	2,194	999	1,297	300	280	280	260	230	(a)	220	230
15	100	1,573	999	1,297	300	280	(a)	260	230	220	220	230
16	95	1,283	999	1,213	300	280	280	260	(a)	220	220	(a)
17	90	856	1,615	1,105	300	(a)	280	260	250	220	220	240
18	90	562	1,611	1,072	300	280	280	260	250	220	(a)	240
19	90	425	1,473	1,260	300	280	280	(a)	250	220	220	240
20	90	294	1,518	1,380	585	280	280	260	250	220	220	240
21	(a)	270	1,811	1,338	704	280	280	275	245	(a)	220	240
22	140	270	1,662	1,260	998	280	(a)	275	250	220	220	240
23	160	270	1,517	1,223	1,325	280	280	275	(a)	220	220	(a)
24	200	270	1,472	1,223	1,422	(a)	280	275	250	220	220	240
25	200	(a)	1,334	1,182	1,301	280	270	275	250	220	(a)	(a)
26	200	564	1,289	869	1,016	280	260	(a)	250	220	220	240
27	200	662	1,206	570	456	280	260	275	250	220	220	240
28	(a)	776	1,206	377	357	280	260	275	245	(a)	220	240
29	220	.....	1,105	(a)	300	280	(a)	275	230	220	220	240
30	220	.....	1,015	300	300	280	260	260	(a)	220	220	(a)
31	220	.....	925	.....	300	.....	260	260	.....	200	.....	240
Mean	136	508	1,365	1,044	562	282	276	264	245	206	199	232

a Water shut back on Sunday when under control.

ANDROSCOGGIN RIVER AT RUMFORD FALLS, MAINE.

This river receives the drainage from the Rangeley and other lakes near the border line between Maine and New Hampshire. It flows in a southerly and southeasterly direction, descending with rapid fall, and is one of the most valuable streams in New England for water-power purposes. It is described in the Nineteenth Annual Report, Part IV, pages 84 to 97. Figures of daily discharge will be found in Water-Supply Papers No. 27, pages 14 to 16. and No. 35, page 28; also

in the Twentieth Annual Report, Part IV, pages 67 to 69. Figures of monthly discharge for the year 1899 will be found in the Twenty-first Annual Report, Part IV, page 57.

The following table of the daily discharge of Androscoggin River for the year 1900 has been furnished by Mr. Charles A. Mixer, resident engineer of the Rumford Falls Power Company, who states that notable features of the discharge of the river for that year (1900) are the high maximum discharge and the high average for February. There is no previous record or recollection of thaws or rains in that section during that month, but in February, 1900, there were two such periods of thawing and rainfall. The precipitation for the month was 7.96 inches, the greatest on record, most of it in the form of rain. In 1898 the precipitation amounted to 7.25 inches, but all in the form of snow, so that the effect on the river was not apparent until spring.

Daily discharge, in second-feet, of Androscoggin River at Rumford Falls, Maine, for 1900.

[Drainage area, 2,320 square miles.]

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	1,231	1,351	3,063	1,830	16,182	5,582	1,780	1,611	1,650	1,985	1,530	3,264
2	1,170	1,295	2,849	1,933	13,249	5,952	1,781	1,560	1,600	1,856	1,466	3,250
3	1,171	1,346	2,791	2,026	12,008	6,850	1,705	1,504	1,650	1,987	1,504	3,059
4	1,190	1,869	2,190	2,561	17,092	7,458	1,640	1,512	1,682	2,137	1,460	3,192
5	1,198	1,362	2,434	2,734	13,700	7,016	1,760	1,500	1,672	2,062	1,479	2,098
6	1,258	1,277	2,136	2,791	10,555	6,196	1,744	1,486	1,603	2,056	1,466	2,173
7	1,224	1,334	2,047	3,592	10,165	6,341	1,791	1,490	1,482	2,079	1,547	2,375
8	1,289	1,354	1,794	4,730	9,121	5,627	1,800	1,505	1,299	2,010	1,694	2,855
9	1,189	1,438	1,810	4,603	8,283	5,550	1,803	1,685	1,264	2,053	10,131	2,090
10	1,115	1,420	1,826	3,836	9,468	4,948	1,798	1,722	1,275	2,006	12,061	1,939
11	1,130	1,654	1,801	3,318	8,024	4,850	1,719	1,620	1,247	2,703	7,525	1,985
12	1,135	1,467	1,768	3,989	7,038	4,509	1,777	1,630	1,273	3,754	5,290	2,339
13	1,211	1,777	1,749	4,138	6,720	4,609	1,781	1,592	1,390	3,197	4,436	2,571
14	1,139	6,009	1,600	4,099	7,734	4,263	1,949	1,648	1,352	2,731	3,870	2,359
15	1,178	5,570	1,772	4,943	10,917	4,113	2,000	1,811	1,332	3,472	3,636	2,470
16	1,147	4,151	1,620	6,230	13,415	4,487	1,880	1,942	1,349	3,615	3,610	2,650
17	1,251	3,599	1,901	8,171	12,062	4,439	2,561	2,009	1,478	3,033	3,104	2,003
18	1,196	2,972	2,549	9,220	13,877	3,015	2,992	1,918	1,667	3,007	3,330	2,071
19	1,181	2,954	2,425	16,130	18,383	2,956	3,168	1,708	1,660	3,255	3,330	2,114
20	1,241	2,562	2,179	22,025	24,531	3,384	3,042	1,714	1,656	2,042	3,898	2,357
21	2,195	2,408	2,491	21,980	22,126	3,311	2,850	1,522	1,658	1,930	6,398	2,215
22	3,221	2,306	2,729	21,317	19,930	2,955	2,885	1,462	1,913	1,509	6,794	2,042
23	2,169	2,146	2,647	21,593	17,632	2,037	2,834	1,437	1,860	1,754	5,819	2,285
24	1,879	2,002	2,255	21,837	16,391	1,948	2,779	1,459	1,860	1,765	4,681	1,764
25	1,765	2,326	1,894	20,250	14,778	1,893	2,844	1,391	1,712	1,676	3,500	2,420
26	1,698	5,157	1,930	17,679	13,408	1,855	2,828	1,318	1,745	1,618	2,934	2,622
27	1,628	4,626	1,819	15,739	12,295	1,855	2,727	1,382	1,697	1,430	3,374	2,300
28	1,382	3,814	1,776	13,458	9,687	1,810	2,530	1,387	2,025	1,430	3,112	1,871
29	1,215	.....	1,836	12,877	9,250	1,931	2,020	1,369	1,930	1,492	2,624	1,983
30	1,285	.....	1,788	13,274	6,973	2,070	1,782	1,413	1,860	1,482	2,834	1,851
31	1,350	.....	1,919	.....	6,632	.....	1,686	1,503	.....	1,500	.....	1,865
Mean	1,408	2,555	2,109	9,763	12,633	4,127	2,201	1,574	1,596	2,213	3,945	2,331

MERRIMAC RIVER AT LAWRENCE, MASSACHUSETTS.

Careful records of the flow of this river at Lawrence, Massachusetts, have been kept for more than fifty years, but they have never been published in full. Figures for the monthly maximum and minimum discharge from 1890 to 1897 were published in the Nineteenth Annual Report, Part IV, pages 113 to 115, and similar figures for 1898 in the Twentieth Annual Report, Part IV, page 73. In Water-Supply Paper No. 35 will be found the figures for the daily discharge for 1897, 1898, and 1899. The figures for 1900 are published herewith.

Daily discharge, in second-feet, of Merrimac River at Lawrence, Massachusetts, for 1900.

[Drainage area, 4,553 square miles.]

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	3,230	4,631	17,350	11,129	10,674	5,332	479	2,134	1,389	2,763	2,456	8,975
2	2,696	4,408	33,802	12,507	11,597	4,356	2,653	2,192	417	2,490	2,468	7,374
3	2,471	2,871	37,732	13,412	11,485	4,397	2,370	2,338	284	2,289	1,608	7,641
4	1,860	2,468	33,090	15,389	12,887	6,520	311	1,505	2,340	2,137	277	6,844
5	1,902	5,520	26,893	16,797	15,881	7,492	2,226	113	2,206	2,109	2,643	11,383
6	1,209	5,687	20,329	17,602	13,223	7,616	2,369	2,038	2,099	1,482	2,478	17,046
7	82	5,910	17,011	19,458	11,654	6,491	1,695	1,694	1,912	246	2,263	15,505
8	2,649	5,933	14,368	21,184	10,332	5,721	350	1,771	1,252	2,315	2,082	12,137
9	2,561	7,112	13,356	21,511	9,817	4,178	2,471	1,860	90	2,515	2,526	9,237
10	2,220	9,550	13,067	18,308	9,661	3,873	2,372	1,939	1,901	2,580	12,221	7,839
11	2,182	9,060	12,211	15,157	10,324	5,275	2,315	1,847	1,637	2,752	12,035	6,101
12	2,327	9,930	11,403	13,660	8,880	3,899	2,339	100	1,540	3,300	9,477	6,020
13	1,463	12,720	11,067	14,497	7,629	3,701	2,270	2,562	1,626	4,406	6,906	5,764
14	266	47,000	10,875	14,557	7,972	3,702	1,629	2,461	1,428	2,914	5,804	5,537
15	2,654	52,989	8,984	13,646	7,001	3,811	186	2,218	732	4,103	5,063	4,338
16	2,574	41,745	10,684	14,643	8,313	2,705	2,022	2,354	37	3,316	4,473	4,023
17	2,299	28,923	13,364	15,945	10,062	2,800	2,206	2,483	1,673	2,904	3,158	5,321
18	2,335	19,311	14,243	17,886	8,772	4,368	2,228	1,986	2,155	3,201	2,466	4,268
19	2,394	13,731	14,536	22,372	7,885	3,526	2,299	1,372	2,242	3,117	4,473	3,692
20	1,893	12,947	16,437	29,896	11,519	2,940	2,283	3,541	2,202	2,283	3,626	3,798
21	2,041	11,549	20,324	34,926	18,169	2,961	1,483	2,469	1,977	890	3,348	4,245
22	6,810	12,317	19,517	30,809	15,538	3,062	146	2,229	1,245	3,248	6,071	3,177
23	7,950	19,300	16,729	27,402	12,433	2,233	2,125	2,072	36	2,702	7,477	3,125
24	8,184	20,400	14,555	24,627	10,344	1,487	2,365	2,072	2,175	2,643	6,235	4,826
25	7,234	24,772	12,680	23,144	9,195	3,915	2,133	1,433	2,385	2,438	4,868	5,034
26	6,027	27,591	12,449	20,338	7,867	2,987	2,159	296	2,229	2,447	7,257	7,615
27	4,401	23,911	11,429	16,730	6,824	2,732	2,189	1,754	2,209	1,568	12,845	7,827
28	3,891	19,187	11,065	14,422	7,271	2,377	1,641	2,030	2,159	961	15,338	6,859
29	6,291	.....	11,264	11,727	6,455	2,448	242	1,958	1,498	2,887	12,689	5,119
30	5,323	.....	11,290	11,326	5,109	1,825	2,487	1,889	309	2,484	10,844	4,139
31	4,776	.....	11,484	.....	6,272	.....	2,317	1,956	.....	2,523	.....	5,473
Mean	3,360	16,481	16,245	18,500	10,034	3,957	1,818	1,876	1,513	2,517	5,849	6,783

NASHUA RIVER AT CLINTON, MASSACHUSETTS.

On the south branch of this stream, near the town of Clinton, the Metropolitan Water Board is now constructing the Wachusett reservoir, for the supply of water to the city of Boston and neighboring cities and towns. Measurements of the flow of Nashua River at Clinton have been made by that board since July, 1896, but the figures have not been published. They have, however, been presented in testimony in court, and have thus been made public. Careful records of rainfall on this watershed have also been made.

SUDBURY RIVER AND LAKE COCHITUATE, MASSACHUSETTS.

Sudbury River and Lake Cochituate have been studied by the engineers of the city of Boston, the State Board of Health of Massachusetts, and the Metropolitan Water Board, and records of rainfall in the Sudbury watershed have been kept since 1875 and in the Cochituate basin since 1852, but the latter have been considered of doubtful accuracy previous to 1872. The records of run-off from 1875 to 1898, inclusive, were published in the Twentieth Annual Report, Part IV, page 75, and those for 1899 in Water-Supply Paper No. 35, page 37, where will also be found a short description of the watersheds of the river and the lake. The following table gives the record for 1900. All of the records were furnished by Mr. Desmond Fitz Gerald.

Run-off of Sudbury River and Lake Cochituate watersheds, in second-feet per square mile, for 1900.

Month.	Sudbury River watershed.	Lake Cochituate watershed.	Month.	Sudbury River watershed.	Lake Cochituate watershed.
January	1.229	1.060	August	−.052	.558
February	5.880	4.186	September	.101	.640
March	5.653	4.020	October	.287	.689
April	2.088	1.454	November	1.026	1.0[illegible]
May	2.081	1.641	December	1.696	1.502
June	.489	.455			
July	−.028	.293	Mean	1.674	1.445

MYSTIC LAKE, MASSACHUSETTS.

This lake has been a source of water supply for the city of Charlestown, Massachusetts, since 1864. A brief description of the watershed is given in Water-Supply Paper No. 35, page 39, and on page 40 the run-off, in inches, for the watershed from 1878 to 1897, inclusive. Records for 1898, 1899, and 1900 are not available, although observations have been continued. The lake is no longer used as a source of water supply.

CONNECTICUT RIVER AT ORFORD, NEW HAMPSHIRE.

This river has its source in Connecticut Lake, in the extreme northern portion of New Hampshire. A gaging and observation station was established August 6, 1900, by E. G. Paul, on the covered highway bridge over the river between Orford, New Hampshire, and Fairlee, Vermont, about 75 miles from the source of the stream. The gage for making observations of the variations in the height of water in the river consists of a scaleboard 20 feet long, graduated to feet and tenths, fastened horizontally to the inside timbers on the upper side of the bridge, 125 feet from the left-bank abutment, and a steel sash chain running over a side pulley with a 5-pound weight, the length of the chain from the extreme end of the weight to the copper-rivet marker being 42.95 feet.

The observer is Frank H. Gardner, of Orford, New Hampshire. One discharge measurement was made during 1900, as follows:

August 7: Gage height, 3.6 feet; discharge, 1,529 second-feet.

Daily gage height, in feet, of Connecticut River at Orford, New Hampshire, for 1900.

Day.	Aug.	Sept.	Oct.	Nov.	Dec.	Day.	Aug.	Sept.	Oct.	Nov.	Dec.
1	.....	3.00	5.30	4.60	8.55	17	6.90	3.05	7.95	6.90	6.[illegible]
2	.....	3.65	6.50	4.50	8.40	18	6.45	2.95	7.60	6.25	6.[illegible]
3	.....	3.70	6.30	4.40	8.00	19	5.60	2.85	7.15	6.50	6.[illegible]
4	.....	3.25	5.20	4.15	7.70	20	5.20	2.85	6.80	7.95	6.[illegible]
5	.....	3.00	4.80	3.85	7.50	21	4.60	2.75	6.50	11.30	6.[illegible]
6	3.90	2.35	4.20	3.80	7.40	22	4.15	3.05	6.25	13.90	6.[illegible]
7	3.65	2.75	5.15	4.30	7.50	23	3.85	4.35	5.65	14.80	6.[illegible]
8	3.75	2.70	6.20	6.00	7.50	24	3.60	5.00	5.75	14.30	6.[illegible]
9	4.05	2.80	5.85	11.25	7.50	25	3.40	4.60	5.65	12.70	7.[illegible]
10	4.85	3.00	6.15	13.40	7.90	26	3.45	4.15	5.75	11.05	9.[illegible]
11	4.95	2.80	7.20	13.35	7.50	27	3.45	3.75	5.70	11.20	8.[illegible]
12	5.10	2.70	12.43	12.40	7.50	28	2.70	3.50	5.45	11.40	8.[illegible]
13	5.35	2.45	12.80	10.35	7.60	29	3.15	4.15	5.40	9.95	8.[illegible]
14	5.75	2.70	10.65	9.55	7.50	30	3.10	4.85	5.00	8.90	.....
15	6.65	2.65	8.90	8.35	7.20	31	3.05	.....	4.75	.....	.....
16	7.50	2.75	8.15	7.95	6.70						

CONNECTICUT RIVER AT HOLYOKE, MASSACHUSETTS.

Measurements have been made of the flow of Connecticut River at Holyoke for many years. For records see Water-Supply Paper No. 35, pages 40 to 42. The figures are furnished by Mr. A. F. Sickman, assistant engineer of the Holyoke Water Power Company. The record for the year 1900 is not available.

CONNECTICUT RIVER NEAR HARTFORD, CONNECTICUT.

Observations of the height of Connecticut River near Hartford are made by the Connecticut River bridge and highway district, Edwin D. Graves, chief engineer, as noted in the Twentieth Annual Report, Part IV, page 77. Daily readings from February 8, 1896, to January 1, 1899, were published in Water-Supply Paper No. 35, pages 42 to 44. The record for 1900 is given in the following table:

Daily gage height, in feet, of Connecticut River near Hartford, Connecticut, for 1900.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	.....	5.7	7.5	.....	12.1	5.7	.....	3.1	2.5	2.0	3.8	7.1
2	4.0	5.6	17.6	7.0	10.7	5.0	2.4	2.9	.....	2.0	3.3	7.0
3	4.0	5.6	18.9	8.0	.....	.....	2.8	2.7	2.0	2.0	3.0	6.6
4	.....	.....	15.1	8.7	10.9	4.8	2.0	2.5	2.5	2.0	3.3	6.3
5	.....	6.9	12.9	8.3	11.2	7.0	2.2	.....	2.5	2.6	3.0	8.0
6	.....	9.8	10.5	9.8	.....	7.5	2.6	1.9	2.5	2.0	2.9	9.3
7	.....	8.7	8.7	10.5	10.2	6.7	2.4	1.9	3.0	2.6	2.9	9.9
8	2.5	7.3	7.8	.....	9.5	5.9	.....	2.4	2.5	2.6	3.3	8.4
9	.....	8.6	7.3	14.2	9.0	5.5	2.3	2.5	.....	2.9	3.6	6.9
10	3.0	10.7	7.3	13.5	8.4	.....	2.4	2.0	.5	3.0	3.0	.....
11	.....	9.4	.....	12.0	8.4	4.7	2.7	2.9	.....	3.0	4.0	4.9
12	4.0	8.5	6.8	10.6	8.4	4.7	2.9	2.0	.....	3.3	5.9	.....
13	.....	11.8	5.9	10.5	.....	4.3	3.0	1.9	2.0	4.0	5.6	.....
14	.....	21.7	6.0	11.2	7.9	4.4	3.0	1.9	2.0	4.6	6.6	.....
15	3.1	23.4	5.8	.....	7.3	4.3	.....	3.4	2.0	5.0	5.9	4.5
16	3.2	22.0	5.4	10.7	6.9	4.0	3.2	3.4	2.0	5.0	5.0	.....
17	3.0	19.0	5.5	12.5	6.7	.....	3.5	3.7	2.0	4.6	4.9	5.3
18	3.0	14.0	.....	14.5	7.2	4.0	3.4	4.5	1.0	4.0	4.3	5.8
19	3.0	10.3	6.0	17.2	8.4	3.9	2.9	.....	1.0	3.6	3.9	5.5
20	3.5	9.3	8.4	20.6	.....	.....	2.6	3.3	1.0	3.6	4.6	5.5
21	.....	8.4	11.2	22.2	13.3	3.4	2.8	3.3	1.3	3.0	3.9	5.2
22	7.8	7.6	10.8	22.8	14.7	3.5	.....	3.0	1.5	3.0	4.6	.....
23	9.9	11.7	9.5	22.3	13.9	3.4	2.6	2.8	1.3	3.6	7.0	.....
24	6.9	11.3	8.9	21.7	13.0	.....	3.0	3.0	1.5	3.0	7.0	5.2
25	6.7	10.5	.....	21.1	11.0	3.0	2.9	2.9	1.6	3.3	7.3	4.8
26	5.7	11.1	7.4	20.5	9.5	3.3	3.4	.....	1.6	3.0	8.5	5.5
27	5.4	9.7	7.3	20.4	.....	3.4	3.4	2.5	1.3	3.0	10.9	6.0
28	6.0	7.8	6.9	17.9	7.4	3.2	3.4	3.0	1.6	3.6	10.1	6.0
29	7.2	.....	6.6	13.9	6.9	3.0	.....	2.9	2.0	3.3	10.2	5.3
30	6.7	.....	6.6	13.6	6.5	3.2	3.3	2.8	1.6	3.6	8.9	.....
31	6.4	.....	6.8	.....	5.9	.....	.....	2.5	.....	3.0	.....	5.7

HOUSATONIC RIVER AT GAYLORDSVILLE, CONNECTICUT.

This gaging station was established October 24, 1900, by E. G. Paul. It is near Merwinsville Station, on the New York and New Haven Railroad. Gage-height observations are made by means of a weight suspended by a sash chain from the covered bridge across the river. The distance from the extreme end of the weight to the copper-rivet marker in the chain is 30.45 feet. The distance from the center of the side pulley to the datum of the gage is 29.35 feet. The gage is referred to a 16-foot scaleboard, graduated to feet and tenths, fastened horizontally to the woodwork on the inside of the covered bridge. The

observer is G. H. Monroe, a farmer living within a short distance of the station. As the cross section of the river channel at the bridge was not favorable for making discharge measurements, a cable was stretched across the stream 1¼ miles below the bridge. The span of the cable is 200 feet. It is supported on the right bank by timbers 25 feet high, and is anchored to a large rock which was placed 5 feet below the surface of the ground. On the left bank the cable is fastened to a large sycamore tree, and is anchored to the base of a large oak tree by means of a turn-buckle. A tag wire with markers every 10 feet was placed above the cable. The initial point of sounding is the large sycamore tree which supports the cable on the left bank.

During 1900 the following discharge measurements were made by Mr. Paul:

August 9: Gage height, 3.30 feet; discharge, 450 second-feet.
August 10: Gage height, 3.25 feet; discharge, 422 second-feet.
October 20: Gage height, 3 feet; discharge, 303 second-feet.
October 24: Gage height, 3.10 feet; discharge, 370 second-feet.

Daily gage height, in feet, of Housatonic River at Gaylordsville, Connecticut, for 1900.

Day.	Oct.	Nov.	Dec.	Day.	Oct.	Nov.	Dec.	Day.	Oct.	Nov.	Dec.
1	.....	3.2	4.9	12	.....	3.7	4.2	23	2.9	3.9	4.[illegible]
2	.....	3.1	4.6	13	.....	3.6	4.2	24	3.1	3.9	5.1
3	.....	3.1	4.3	14	.....	3.7	4.2	25	3.2	3.8	4.3
4	.....	2.9	4.2	15	.....	3.7	Ice.	26	3.1	4.6	4.2
5	.....	3.0	5.6	16	.....	3.7	Ice.	27	3.1	5.5	4.1
6	.....	2.9	5.8	17	.....	3.6	Ice.	28	3.1	5.4	4.2
7	.....	2.8	5.4	18	.....	3.5	Ice.	29	3.1	5.1	4.0
8	.....	3.2	5.1	19	.....	3.6	Ice.	30	3.0	4.9	3.9
9	.....	3.5	4.9	20	.....	3.7	Ice.	31	.....	.....	4.2
10	.....	4.1	4.5	21	.....	4.1	Ice.				
11	.....	3.9	4.2	22	.....	4.0	Ice.				

MISCELLANEOUS MEASUREMENTS OF STREAMS NEAR GREATER NEW YORK.

Measurements of the discharge of three small streams near the city of New York were made by E. G. Paul in 1900. These streams—Tenmile River, Wallkill River, and Esopus Creek—have been under consideration as possible sources for increasing the water supply of Greater New York. Following are the discharge measurements made by Mr. Paul:

Miscellaneous measurements of streams near Greater New York.

Date.	Stream.	Locality.	Discharge.
1900.			Sec.-ft.
Aug. 9	Tenmile River	Webatuck, N. Y.	46
Oct. 20	do	do	32
Aug. 18	Wallkill River	Rockford Bridge, N. Y.	70
Aug. 14	Esopus Creek	Brodheads Bridge, N. Y.	92

METHODS EMPLOYED IN THE GAGING OF NEW YORK STREAMS DURING THE YEAR 1900.[1]

The location of gaging stations which have been maintained or established in New York State during the year 1900 are given in the first of the tables which follow. Most of the older stations were established either in connection with the Upper Hudson storage survey of 1895 and 1896 or by the United States Board of Engineers on Deep Waterways in 1898. All of the older stations are in connection with dams and mills. The methods employed in computing the flow of streams from such records are described in Water-Supply Paper No. 35, page 39, and records of flow for past years will be found in the same paper. In connection with all such stations an effort has been made to improve upon the methods formerly used, and to check the previous results by making current-meter measurements to determine the volume of flow through turbines, and the proper allowance to be made for leakage of dams, also to check the calculated flow over dams.

Measurements of streams in New York State have been made with the cooperation of the State engineer and surveyor, Edward A. Bond, and the deputy State engineer and surveyor, William Pierson Judson. The stations established during the year are chiefly current-meter stations, a gage being read once or twice daily to determine the stage of the river, and the current-meter measurements made, as opportunity permits, with the water at different stages. A sufficient number of current-meter measurements have not yet been made at any of the stations to permit the construction of rating curves, so that the daily flows in second-feet can not yet be given.

In addition examinations have been made of various streams, notably of those flowing from the northern slope of the Adirondack region, to determine favorable localities for establishing gaging stations, and a number of gaging records have been obtained which are not yet ready for publication.

No single method of gaging has been followed exclusively, but different methods have been pursued, with a view to checking the results obtained. In a number of instances weir and current-meter measurements have been combined at a single station, the former method being employed to calculate the flow over dams, the latter method to estimate diversion to canals or for water-power purposes.

Many of the older stations had been neglected, and at the beginning of the year the gages were out of repair. Many of the gages have been replaced by new ones having metallic figures and division marks, and other changes have been made with a view to increasing the accuracy of the results.

[1] Report of Robert E. Horton, under whose direction the measurements in New York State have been made.

Location of gaging stations on New York streams.

Stream.	Location.	Kind of station.	Duration of record.	Gage reader.
Mohawk River	Ridge Mills	Dam and waterworks.	Oct. 1, 1898, to Nov. 28, 1900.	Daniel Brown.
Oriskany Creek	Oriskany	Dam and feeder	Oct. 16, 1898, to Jan. 31, 1901.	Frank Baker.
Sauquoit Creek	New York Mills (No. 3).	Dam and mill	Sept. 20, 1898, to Sept. 30, 1900.	Robert Hughes.
West Canada Creek.	Twin Rock Bridge.	Bridge	Sept. 9, 1900—Cont'd	Utica Elec. Lt. and Power Co.
Do	Middleville	Dam and mill	Oct. 7, 1898—Cont'd *a*	E. J. Nelson.
Mohawk River	Little Falls	Dam and mills	Sept. 23, 1898—Cont'd	J. J. Gilbert and Wm. Hoffman.
East Canada Creek.	Dolgeville	Dam and power plant.	Sept. 23, 1898—Cont'd	Henry Meyer.
Cayadutta Creek.	Below Johnstown.	do	Oct. 1, 1898—Cont'd	A. N. Terry.
Schoharie Creek	Fort Hunter	Dam and feeder.	Sept. 23, 1898, to July 31, 1900. *b*	H. J. Wittemeier.
Do	Erie Canal aqueduct.	Bridge	May 2, 1900, to Oct. 13, 1900.	James Shutts.
Do	Mill Point	do	July 6, 1900—Cont'd	Henry Peters.
Do	Schoharie Falls	Dam and plant	June, 1900—Cont'd	Empire State Power Co.
Mohawk River	Near Schenectady.	Bridge	Feb. 1, 1899—Cont'd *c*	L. Diggins.
Do	Rexford Flats	Dam and feeder.	Dec. 8, 1898—Cont'd	H. R. Betts.
Do	Dunsbach Ferry.	Dam	Mar. 12, 1898—Cont'd *d*	Kept for D. J. Howell, C. E.
Indian River	Indian Lake dam	Storage dam	July 22, 1900—Cont'd.	Frank Pelon
Schroon River	Below Warrensburg.	Dam and mill	Nov. 1, 1895—Cont'd	Joseph Goodfellow.
Hudson River	Fort Edward	Dam and mills	Dec. 1, 1895—Cont'd	B. A. Carr.
Do	Mechanicsville	do	Dec. 1, 1897—Cont'd	The Duncan Co.
Seneca River	Baldwinsville	do	Nov. 12, 1898—Cont'd.	Chas. Brannock.
Chittenango Creek.	Bridgeport	do	Sept. 16, 1898—Cont'd	Jefferson Downs.
Oneida Creek	Kenwood	Dam and mill	Oct. 4, 1898, to Dec. 31, 1900. *e*	Wm. Padgham.
Fish Creek, West Branch.	McConnellsville	Dam and mills	Sept. 13, 1898—Con'd *f*	Frank S. Harden.
Oswego River	High dam	do	Apr. 1, 1897—Cont'd	Oswego Waterworks Co.
Do	Above Minetto	Cable	Sept. 14, 1900—Cont'd	H. L. Woodcock.
Salmon River	Above Pulaski	Bridge	Sept. 4, 1900—Cont'd	H. A. Walker.
Moose River	Moose River	Cable	June 5, 1900—Cont'd	Frank W. Smith.
Black River	Huntingtonville dam.	Waterworks dam.	Feb. 22, 1897—Cont'd	Alonzo Dressor.

a No record February 1 to May 9, 1900, inclusive.
b No record December, 1899, and February, 1900.
c No record October 3, 1899, to April 7, 1900, inclusive.
d No record April 1, 1899, to August 31, 1900, inclusive.
e No record August and September, 1899, and February and May, 1900.
f No record August, 1899, to April, 1900.

In the foregoing table there have been included a number of rivers which belong to the Great Lakes drainage, descriptions of which will be found in Water-Supply Paper No. 49, the same being Part III of this series, a geographic arrangement of the streams having been followed, as in former years. The streams referred to are Seneca River at Baldwinsville, Chittenango Creek at Bridgeport, Oneida Creek at Kenwood, West Branch of Fish Creek at McConnellsville, Oswego River at Minetto and at high dam, Salmon River above Pulaski, Moose River below McKeever, Beaver River, and Black River at Huntingtonville.

The following table gives the results of current-meter measurements made on the principal of these streams during the year 1900. A modified form of the Price meter which has been adopted by the United States Geological Survey was used. The usual mode of procedure was to submerge the meter six-tenths of the depth of the stream, at measured intervals across the channel, and record the revolutions for a period of 100 seconds. In cases of doubt, surface and bottom

velocities were also taken, or, as a check, the flow was determined by the integrating method. Many of these measurements were made at places where permanent stations have not been maintained, and the gage-height figures given are the distances to water surface measured from some fixed point of reference, usually the coping of either a pier or an abutment. In addition to the results included in the table, 35 current-meter measurements have been made in headraces, feeders, power canals, and other channels, to determine the leakage of dams, the flow through turbines, and the amount of diversions. The results of these measurements will be found in connection with the descriptions of the various gaging stations.

Summary of current-meter measurements of New York streams during 1900.

Date.	Stream.	Location.	Point of measurement.	Hydrographer.	Gage height.	Discharge.
					Feet.	*Sec.-ft.*
May 29	Mohawk River	Riverside Bridge	Two miles above Rome.	R. E. Horton	14.90	202
Aug. 31	do	Rome	Below Erie Canal feeder dam.	do		38
Do	do	do	Above Erie Canal feeder dam.	do		188
Apr. 22	do	Ridge Mills	At highway bridge above dam.	H. A. Pressey	1.96	1,385
Aug. 23	do	do	In channel below dam.	R. E. Horton		116
Apr. 30	Oriskany Creek	Oriskany	Wood road bridge	do	12.20	289
Sept. 7	West Canada Creek.	Twin Rock Bridge.		do	.37	182
Sept. 10	do	Middleville		do		245
May 1	Mohawk River	Little Falls	Suspension bridge	do	18.70	4,773
May 23	do	do	Astronga Bridge	do	1.33	1,569
May 22	East Canada Creek.	Dolgeville	Iron highway bridge	do	20.12	412
July 27	do	do	do	do	20.16	452
July 29	do	do	Below High Falls dam.	do		108
Apr. 24	Schoharie Creek	Fort Hunter	West Shore railroad bridge.	H. A. Pressey		5,573
May 2	do	do	Erie Canal aqueduct	E. D. Walker	2.26	1,257
June 21	do	do	Inflow to Mohawk	R. E. Horton		30
July 18	do	do	Below aqueduct; inflow to Mohawk.	E. D. Walker		38
Do	do	do	Total outflow at dam.	do		114
Aug. 22	do	do	Below aqueduct; inflow to Mohawk.	do		44
July 5	do	Mill Point		do	.64	87
Aug. 22	do	do		do	.47	141
May 12	Mohawk River	Schenectady	Freeman's bridge	do	6.50	4,135
July 17	do	do	do	do	5.26	667
Aug. 21	do	do	do	do	5.40	976
Oct. 19	Indian River	Sabael	Below Indian Lake dam.	R. E. Horton	8.20	451
July 25	Schroon River	Warrensburg	Two miles above Warrensburg.	do	16.40	383
Aug. 9	do	do	One mile above mouth.	do		236
July 26	Hudson River	Fort Edward	Highway bridge below paper mill.	do		2,704
Oct. 20	do	Mechanicsville	Highway bridge below Duncan & Co.'s dam.	do	21.50	1,871
June 11	Seneca River	Baldwinsville		do		1,863
June 16	Chittenango Creek.	Bridgeport	Highway bridge below Butternut Creek.	do		95
Sept. 17	Oneida Creek	Kenwood	Headrace above mill.	do	3.25	20
June 1	do	Oneida Castle		do	8.50	36
May 17	Fish Creek, East Branch.	Point Rock		do		485
Sept. 15	Oswego River	Cable station	Three miles above Minetto.	do	5.40	1,677
Sept. 4	Salmon River	Bridge station	Two miles above Pulaski.	do	1.08	103
June 6	Black River	Glen Park Bridge	Two miles below Watertown.	do		2,175

Field work was begun so late that in many instances current-meter measurements to check the calculated flow over dams during high water could not be made. Additional meter measurements are also needed at all of the current-meter stations to establish discharge curves. This report must therefore be considered as in some degree preliminary.

For mill streams, where the water is held back as pond storage during the dry season, and where the mills are stopped on Sundays or holidays, it is impossible to determine the natural regimen of flow. If at the time the water wheels are stopped the water stands below the level of the crest of the dam, the flow in the stream channel below will be nil, or at best will only equal the leakage of the dam, flumes, or penstocks. With regard to estimation of Sunday flow, no uniform rule has been followed. In some of the older records the Sunday flow during the dry season has been taken as the mean of the calculated flow for the preceding and following days, and in cases where this method had previously been used it has been adhered to in computing the daily discharge of the streams. In other instances the daily flow given in the table is that shown by the gaging record, and represents as nearly as may be the actual amount of water flowing in the stream channel below the dam, but it may be quite different from the amount entering the pond above the dam.

The relation existing between the canals of New York and the streams of the central portion of the State is very complicated. In many cases diversion from the headwaters of the streams for the supply of canals virtually reduces their effective drainage areas. As a result, the watersheds during the summer may be materially less in area, and in their water-yielding characteristics may differ widely from the region tributary to the streams when the canals are not in operation. It is evident that the run-off from such streams is not comparable with that from streams having an undisturbed regimen.

It often happens that a single gage reading taken at or near the culmination of a flood shows a larger flow than the mean for any single day. The results of such isolated observations, together with other data relative to extremes of flow, have been given for a number of stations.

The drainage areas of the various streams above the gaging stations and at other points are given in the following table:

Drainage areas of New York streams.

Stream.	Location.	Area.
		Sq. miles.
Batten Kill *a*	At mouth	*b* 438
Beaver River	do	322
Do	Above Beaver	153
Do	Below Beaver	169

a Hudson River tributary. *b* Upper Hudson storage surveys.

Drainage areas of New York streams—Continued.

Stream.	Location.	Area.
		Sq. miles.
Black Creek *a*	At mouth	39
Black River	do	1,930
Do	At gaging station	1,889
Do	At Forestport	*b* 268
Cayadutta Creek *c*	At mouth	*b* 62
Do	At gaging station	*b* 40
Chittenango Creek	At mouth	*b* 309
Do	At Bridgeport *d*	*b* 307
East Canada Creek *c*	At mouth	283
Do	At gaging station	256
Fish Creek, East Branch	At Point Rock	*b* 104
Fish Creek, West Branch	At McConnellsville *d*	*b* 187
Fulton Chain	Above Old Forge	41
Hoosic River *e*	At mouth	*f* 730
Hudson River	At Troy	*f* 8,000
Do	Above Mohawk	4,627
Do	At Mechanicsville	*f* 4,500
Do	At Fort Edward	*f* 2,800
Do	At Hadley *g*	*f* 1,092
Indian Lake *e*	At gaging station	146
Lake Neatahwanta *a*	At mouth of outlet	23
Mohawk River	At mouth	*b* 3,468
Do	At Rexford Flats *d*	*b* 3,385
Do	At Schenectady *d*	3,212
Do	At Little Falls *d*	*b* 1,306
Do	At Ridge Mills *d*	*b* 153
Moose River	At mouth	406
Do	At cable station	346
Ninemile Creek *c*	At mouth	*b* 74
Do	At gaging station	*b* 63
Oneida Creek	At mouth	*b* 149
Do	At Kenwood *d*	*b* 59
Oneida River	At mouth	*b* 1,402
Oriskany Creek *c*	do	*b* 146
Do	At State dam *d*	*b* 144
Oswego River	At mouth	5,002
Do	At high dam	5,000
Do	At cable station	4,990
Do	At Fulton	4,916
Do	Below Three River Point	*b* 167
Sacandaga River *e*	At mouth	*b* 1,056
Salmon River	At bridge station	264
Do	One mile above falls	*b* 191
Sauquoit Creek *c*	At mouth	67
Do	At gaging station	52
Schoharie Creek *c*	At mouth	*b* 947
Do	At Fort Hunter *d*	*b* 947
Do	At Mill Point *d*	934
Do	At Schoharie Falls *d*	930
Schroon River	At gaging station	563
Seneca River	At mouth	*b* 3,483
Do	At Baldwinsville *d*	*b* 3,103
West Canada Creek *c*	At mouth	569
Do	At Middleville *d*	518
Do	At Trenton Falls	375
Do	At Twin Rock Bridge *d*	252

a Oswego River tributary.
b United States Board of Engineers on Deep Waterways.
c Mohawk River tributary.
d Gaging station.
e Hudson River tributary.
f Upper Hudson storage surveys.
g Not including Schroon River.

MOHAWK RIVER GAGING STATIONS.

Gaging records have been kept on this stream at the following dams and mills: Rome waterworks pumping station at Ridge Mills, lower dam at Little Falls, New York State dam at Rexford Flats, and West Troy Company's dam at Dunsbach Ferry. A current-meter station at Freeman's bridge, below Schenectady, has also been maintained.

An important series of gagings has also been instituted on Mohawk River by the New York State canal survey, under the direction of Edward A. Bond, State engineer and surveyor. A statement regarding these gagings has been furnished by Mr. D. J. Howell, consulting engineer of the New York State canal survey. Gages have been erected at various places along the stream, from Herkimer to the confluence of the river with Hudson River at Cohoes, both above and below each dam and near the points of inflow of the more important tributaries, and the positions of the gage zeros have been determined with reference to mean tide as a datum. Gages previously maintained by the United States Geological Survey have been used wherever available. Observers are employed to take simultaneous daily readings of the gages, from which the slope of the water surface for each level or division will be determined and the velocity of flow computed. It is the intention to take cross sections of the stream channels in each level, at different stages, and to make current-meter or other discharge measurements during high water.

The regimen of Mohawk River during the navigation season is undoubtedly influenced to a large extent by the Erie Canal, which runs parallel to it from Rome to Cohoes. The water supply of the Erie Canal east of the summit level at Rome is, with a single exception, derived from Mohawk River and its tributaries. The State dams and feeders are located as follows:

Location of State dams and feeders.

Stream.	Location.
Mohawk River	Delta, 6 miles above Rome.
Do	Rome.
Oriskany Creek	Oriskany.
Mohawk River	Little Falls.
Do	Fivemile or Rocky Rift dam.
Schoharie Creek	Fort Hunter.
Mohawk River	Rexford Flats.

A large diversion from the watershed at these feeders is in some measure counterbalanced by return water to the main channel, from seepage through canal and feeder banks, and from flow over waste-weirs.

The gaging records at Rexford Flats and at Little Falls indicate that during the navigation season the yield of the watershed, in

second-feet per square mile, and frequently, also, the actual flow of the river, in second-feet, is considerably less at the former station than at the latter station. The drainage area above Rexford Flats is 3,385 square miles, or 2.6 times that at Little Falls, which is 1,306 square miles. This fact appears to be confirmed by the other gaging records kept on the stream, but the results of these have not been sufficiently worked up to permit a final discussion of the subject at this time. The diminished water-yielding capacity of the lower Mohawk Basin may be attributed in part to the low water of Schoharie Creek. The total drainage area of that creek is 947 square miles. Weir measurements at Schoharie Falls show that the flow sometimes falls below 50 second-feet. During practically the entire summer no water flows over the crest of the State dam at Fort Hunter; the major portion of the flow is diverted to the Erie Canal feeder, and the remainder leaks through the dam. During the summer of 1900, from June to October, inclusive, the direct inflow to the Mohawk from this tributary did not, with the exception of a few days, exceed 45 second-feet, or 0.05 second-foot per square mile.

MOHAWK RIVER AT RIDGE MILLS, NEW YORK.

A description of this station will be found in Water-Supply Paper No. 35, page 45. During the present season (1900) the calculated discharge of the turbines has been made to depend on current-meter measurements, instead of on the observed wheel-gate openings, as formerly.

Table showing relation of speed of pumps to water flowing in tailrace.

Date.	Speed of pumps.	Measured flow in tailrace.
1900.	*Rev. per min.*	*Second-feet.*
May 29	15	122
August 23	12	95

The discharge of the turbines is sensibly proportional to the rate at which the waterworks pumps which they drive are run, and a straight-line diagram has been prepared, using the foregoing data, from which the flow through the turbines has been estimated.

On August 23, when no water was flowing over the crest, a measurement of the leakage of the dam was made in the stream channel below; it was found to be 20 second-feet, and an allowance of that amount has been made in estimating the daily flow. The results of other meter measurements at this station, and in the vicinity, will be found in the table on page 39.

The gaging record at Ridge Mills does not include any allowance for

diversion to Black River Canal at the Delta feeder, 4 miles upstream, nor for return water from seepage and wasteweirs.

Daily discharge, in second-feet, of Mohawk River at Ridge Mills, New York, for 1898.

[Drainage area, 153 square miles.]

Day.	Oct. *a*	Nov. *a*	Dec. *a*	Day.	Oct. *a*	Nov. *a*	Dec. *a*
1	154	289	123	18	215	294	169
2	149	249	144	19	199	294	144
3	138	215	133	20	249	304	159
4	127	199	133	21	215	279	342
5	143	215	169	22	999	369	609
6	249	249	184	23	859	697	974
7	199	369	144	24	794	409	639
8	173	259	169	25	519	294	419
9	159	249	169	26	609	215	344
10	149	1,134	154	27	1,251	191	319
11	143	2,134	184	28	574	199	159
12	149	697	184	29	409	134	219
13	154	515	121	30	539	104	324
14	259	439	136	31	314	.....	644
15	889	371	159				
16	349	359	159	Mean	369	401	261
17	319	315	154				

a Revised figures.

Daily discharge, in second-feet, of Mohawk River at Ridge Mills, New York, for 1899.

Day.	Jan. *a*	Feb. *a*	Mar. *a*	Apr. *a*	May. *a*	June.	July.	Aug.	Sept.	Oct.	Nov. *a*	Dec. *a*
1	316	214	351	364	321	337	262	218	.....	93	515	225
2	243	214	394	319	319	319	252	233	.....	203	515	695
3	296	214	366	254	239	279	232	248	.....	233	345	395
4	298	214	306	311	204	259	249	131	.....	253	295	345
5	2,373	199	176	344	176	214	202	146	.....	200	365	295
6	896	164	1,074	391	151	232	127	131	.....	310	295	295
7	409	123	735	439	102	232	262	126	.....	290	265	275
8	316	166	414	1,264	94	262	322	121	.....	260	295	235
9	299	126	346	1,034	129	259	669	125	85	290	295	235
10	219	129	219	686	138	279	499	129	85	310	245	235
11	164	129	254	799	344	249	379	139	85	340	315	315
12	157	129	1,174	1,364	414	282	359	169	95	260	295	3,625
13	157	129	1,211	1,499	296	282	339	265	89	310	265	1,155
14	153	173	701	2,181	361	302	339	245	85	260	265	519
15	911	265	364	1,744	214	402	319	275	99	310	265	455
16	725	258	321	1,841	161	399	399	295	115	240	265	365
17	459	184	239	1,364	389	359	319	230	135	220	295	295
18	324	126	219	1,654	361	339	399	230	112	290	295	295
19	223	166	274	2,226	549	282	382	245	85	290	265	1,075
20	199	151	581	2,034	614	279	339	245	79	290	245	1,495
21	193	160	464	1,404	436	262	302	265	79	240	245	1,086
22	208	494	364	1,264	319	259	382	295	56	260	265	315
23	214	697	389	1,139	274	299	299	272	56	260	265	315
24	261	389	436	959	239	259	282	289	56	310	265	345
25	338	346	416	766	219	259	275	262	60	540	295	315
26	318	296	344	614	204	259	262	269	69	220	265	295
27	244	436	274	549	189	249	289	309	69	310	265	295
28	223	451	241	439	386	249	165	309	53	340	265	295
29	208	.....	536	354	1,136	249	133	278	53	360	205	295
30	170	.....	436	319	546	229	178	315	79	260	205	275
31	170	.....	436	.....	344	.....	178	200	.....	260	.....	275
Mean	377	244	467	997	320	281	310	226	81	278	291	532

a Revised figures.

Daily discharge, in second-feet, of Mohawk River at Ridge Mills, New York, for 1900.

Day.	Jan.a	Feb.	Mar.	Apr.	May.b	June.b	July.b	Aug. b	Sept.b	Oct.b	Nov.
1	270	200	784	810	130	168	125	104	155	175	395
2	270	132	815	1,080	130	216	115	120	115	185	365
3	270	100	750	1,245	130	285	105	120	109	165	340
4	290	180	630	1,135	130	230	105	92	105	113	340
5	310	100	280	1,065	130	180	160	90	112	115	460
6	310	248	224	1,245	130	168	158	104	112	113	1,970
7	340	128	354	2,205	130	215	175	84	112	115	965
8	1,115	125	359	1,845	130	180	170	84	104	111	835
9	475	1,628	320	1,070	130	230	112	107	85	113	705
10	530	1,128	300	740	130	230	110	100	100	127	725
11	475	854	280	680	130	185	120	88	100	118	725
12	420	738	219	930	125	180	150	154	107	118	640
13	415	3,162	254	930	100	180	137	365	92	109	620
14	305	1,928	254	835	100	215	130	458	95	102	645
15	305	1,058	219	900	100	165	100	364	115	395	580
16	265	618	164	1,485	105	155	100	88	115	395	580
17	265	578	149	1,685	210	158	160	84	215	395	620
18	345	284	159	3,375	210	180	140	80	325	109	640
19	365	202	169	1,905	147	165	163	84	118	90	1,550
20	1,365	188	544	935	147	170	165	92	179	90	1,760
21	1,535	198	590	930	130	170	250	98	625	95	1,480
22	875	288	594	1,235	130	165	185	124	615	95	965
23	655	803	594	930	112	185	125	97	415	95	965
24	285	714	370	675	105	205	132	84	435	895	735
25	365	268	164	555	105	140	515	165	395	415	1,665
26	480	172	160	365	105	145	270	165	205	395	3,990
27	285	202	164	365	105	123	198	120	175	410	(c)
28	265	132	160	315	105	180	175	185	127	395	-------
29	270	-------	164	275	110	130	180	125	185	345	-------
30	270	-------	116	225	105	124	107	124	205	95	-------
31	270	-------	112	-------	105	-------	117	179	-------	95	-------
Mean	160	581	336	1,062	126	180	160	140	198	212	971

a Record doubtful; owing to ice on crest of dam.
b Record doubtful; flashboards changed frequently.
c Dam and gage injured in flood.

ORISKANY CREEK AT ORISKANY, NEW YORK.

A description of this station, as well as of a second station which was maintained for a time at Coleman, on the same stream, will be found in Water-Supply Paper No. 35, page 47. The Oriskany station is located at the New York State dam. During the summer the entire flow, less leakage, is ordinarily diverted to the canal feeder. H. Waterbury & Company's dam, located just below the State dam, backs water above the toe of the latter dam, so that direct measurements of the leakage of the State dam can not readily be made. During the winter and spring the flow of the river is available for power from the lower dam, but during the season of navigation the inflow to the river from this tributary amounts to only a few second-feet. The computed flow at the gaging station represents the total outflow from the pond above the State dam, and includes water diverted from Chenango River through the channel of Oriskany Creek to feed Erie Canal. A record is kept of the height of water in the pond above the dam, and also of that in the feeder channel below the head gates. The observed difference, or the head on the feeder gates, together with the area of the gate openings, have been used in the formula for discharge through submerged orifices to determine the flow. A screen rack in the forebay, just above the feeder gates, often becomes clogged with drift, causing a loss of head of several inches. In order that the correct head on the feeder gates might be

obtained, at the beginning of the navigation season of 1900 a gage was placed in the forebay between the screen rack and the feeder gates.

Current-meter measurements have been made in the Oriskany feeder, as follows:

Current-meter measurements in Oriskany feeder.

Date.	Hydrographer.	Measured discharge.	Computed discharge.
		Second-feet.	*Second-feet.*
October 15, 1898	W. D. Lockwood	49.5	48.9
Do	do	118.9	119.1
April 28, 1900	R. E. Horton	167.9	170.3
May 29, 1900	do	103.2	

During the dry season the gateways leading to the feeder are wide open and the water flows through unobstructed, as in an open channel, so that the formula for discharge through orifices can not be applied.

In this connection the difficulties encountered in gaging the flow in canal feeders are worthy of comment. Broadly speaking, the amount of water required for the supply of canals is proportional to lockage and evaporation jointly, with, perhaps, a constant factor added for seepage losses. As a matter of fact, however, the rate of flow in the feeder often fluctuates, within wide limits, several times a day. Usually gates are placed in both the inlet and the outlet ends of the feeder channel. The height of the water in the feeder is influenced by the height of the water in the canal and in the supply pond above, while the velocity of flow may be varied by changes in the gate openings at either end. Isolated discharge measurements are of value in a general way, but it may be said that nothing short of a continuous record, both of the stage of the water in the feeder and of its velocity of flow, will serve to determine the actual diversion from day to day.

Daily discharge, in second-feet, of Oriskany Creek at Oriskany, New York, for 1898.

[Drainage area, 144 square miles.]

Day.	Oct.	Nov.	Dec.	Day.	Oct.	Nov.	Dec.
1		329	266	18	212	304	160
2		298	259	19	231	310	220
3		274	428	20	266	302	195
4		276	413	21	248	359	310
5		266	457	22	328	263	340
6		269	445	23	350	300	835
7		267	445	24	316	324	410
8		236	462	25	230	278	355
9		284	393	26	330	253	210
10		370	388	27	564	502	150
11		740	425	28	457	335	160
12		370	460	29	408	309	155
13		365	346	30	339	254	155
14		333	160	31	336		265
15		352	150				
16	338	311	300	Mean	325	327	2[illegible]
17	248	304	280				

Daily discharge, in second-feet, of Oriskany Creek at Oriskany, New York, for 1899.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	180	157	495	180	238	90	.....	206	83	180	320	90
2	180	167	355	170	144	80	.....	206	87	138	175	103
3	195	150	425	160	139	80	.....	167	73	108	115	73
4	220	177	460	260	139	80	.....	126	10	99	138	108
5	220	167	910	390	129	80	.....	126	75	119	144	80
6	250	217	650	425	124	80	138	177	108	132	346	107
7	285	230	410	525	122	80	184	177	101	122	374	99
8	220	243	300	1,270	122	80	170	195	133	138	429	40
9	260	277	240	770	124	80	104	195	176	95	364	30
10	220	277	215	600	124	80	110	195	180	58	394	10
11	255	343	175	490	28	89	117	195	159	106	634	45
12	255	343	645	1,220	28	94	124	195	95	97	534	340
13	410	364	490	1,440	25	89	124	195	73	92	564	330
14	505	343	340	1,160	25	94	124	195	87	26	584	80
15	550	364	280	910	30	104	124	206	85	72	564	65
16	505	337	255	625	105	101	206	206	89	45	484	80
17	380	343	200	370	122	94	208	206	115	106	524	40
18	315	337	230	340	134	113	197	206	123	58	584	65
19	255	304	355	290	139	113	183	206	119	71	202	250
20	260	297	370	260	120	113	183	206	138	48	53	130
21	290	390	330	230	140	113	270	206	161	82	124	110
22	270	410	285	130	160	113	270	206	129	73	154	50
23	290	404	355	50	140	118	270	206	162	85	174	50
24	290	303	280	50	115	118	196	206	102	88	214	70
25	260	287	285	50	115	118	196	206	112	70	174	50
26	285	297	240	416	159	118	196	194	181	70	438	40
27	305	367	220	406	144	118	196	184	222	54	318	30
28	340	244	215	120	139	113	196	184	216	158	318	35
29	305	.....	205	335	139	113	196	184	185	72	582	30
30	290	.....	190	325	129	113	196	98	206	55	789	65
31	300	.....	190	.....	134	.....	196	98	.....	115	.....	70
Mean	295	291	342	466	119	99	180	186	126	91	360	89

Daily discharge, in second-feet, of Oriskany Creek at Oriskany, New York, for 1900.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	64	139	254	1,318	238	95	110	100	82	70	186	250
2	133	212	786	1,391	192	95	110	100	74	70	191	205
3	81	326	876	1,046	163	95	110	100	76	70	136	185
4	96	286	776	546	163	95	110	100	84	70	117	175
5	41	416	736	426	144	95	110	100	66	70	122	607
6	76	302	398	916	183	95	140	100	81	70	73	427
7	54	218	756	896	161	95	88	100	85	70	95	330
8	101	302	454	596	163	97	73	100	85	175	101	285
9	54	1,871	366	308	136	95	104	100	61	66	105	205
10	114	596	302	206	133	95	101	100	73	63	35	177
11	56	416	218	194	150	95	107	100	73	75	185	202
12	96	139	163	286	70	95	105	100	61	75	151	147
13	51	3,116	157	316	105	95	105	100	70	75	106	292
14	68	454	157	254	94	95	105	124	70	75	113	142
15	81	248	139	212	105	95	105	107	70	78	136	112
16	133	139	133	206	118	95	105	103	70	84	96	157
17	145	114	127	470	144	95	105	100	70	81	113	202
18	114	76	76	1,090	171	95	105	100	70	78	114	217
19	546	60	170	427	156	95	107	100	70	80	81	217
20	1,376	81	696	315	148	95	105	100	70	80	127	337
21	576	127	496	212	161	95	107	100	70	75	127	277
22	721	170	294	877	115	95	124	100	70	75	78	242
23	176	236	326	379	113	95	105	100	70	77	158	272
24	91	96	386	260	113	95	105	100	70	83	161	1,172
25	481	91	254	239	113	95	114	100	70	75	430	427
26	156	56	236	235	113	95	110	100	70	75	2,592	242
27	51	170	183	201	113	95	112	110	70	103	857	217
28	106	133	227	283	113	97	108	132	75	100	342	172
29	91	.....	286	282	113	95	33	111	85	100	242	172
30	114	.....	686	272	113	95	32	100	85	100	297	192
31	156	.....	846	.....	113	95	30	100	.....	191	.....	177
Mean	199	378	386	488	136	95	100	103	73	85	255	272

SAUQUOIT CREEK AT NEW YORK MILLS, NEW YORK.

A description of this station will be found in Water-Supply Paper No. 35, page 48. During the summer little water flows over the dam, the entire volume being used to drive the water wheels in the adjoining cotton mills. The leakage of the dam was measured by current meter on May 31, and was found to be 5.6 second-feet.

This station was discontinued October 1, 1900.

Daily discharge, in second-feet, of Sauquoit Creek at New York Mills, New York, for 1898.

[Drainage area, 52 square miles.]

Day.	Sept.	Oct.	Nov.	Dec.	Day.	Sept.	Oct.	Nov.	Dec.
1	.....	16	42	49	18	.....	105	49	*30
2	.....	*7	43	44	19	.....	60	43	42
3	.....	27	38	34	20	18	36	*95	36
4	.....	18	36	*50	21	26	38	46	76
5	.....	31	32	46	22	18	42	61	194
6	.....	54	*59	42	23	16	*73	69	28
7	.....	35	38	44	24	22	71	76	39
8	.....	23	36	49	25	*37	48	42	*76
9	.....	*23	38	42	26	30	59	66	62
10	.....	35	132	28	27	40	140	*62	46
11	.....	52	140	*22	28	40	74	36	44
12	.....	46	58	42	29	30	68	43	34
13	.....	46	*69	42	30	21	*66	42	74
14	.....	46	62	36	31	.....	57	.....	52
15	.....	144	52	37					
16	.....	*80	49	42	Mean	27	56	57	57
17	.....	105	49	34					

* Sunday.

Daily discharge, in second-feet, of Sauquoit Creek at New York Mills, New York, for 1899.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	*72	42	77	45	56	42	12	17	18	(*)	53	19
2	158	36	48	*40	50	34	(*)	11	5	29	42	13
3	58	38	48	57	48	18	27	43	(*)	24	21	*23
4	111	18	42	73	36	*30	10	30	.....	21	26	22
5	140	*40	*480	104	33	42	30	12	33	15	*29	21
6	74	36	109	165	28	56	27	(*)	32	16	32	26
7	50	36	72	188	*26	48	18	26	6	8	22	26
8	*13	26	73	47	47	39	16	25	21	*13	24	19
9	58	30	62	*156	35	23	*25	22	12	18	24	12
10	38	21	60	100	26	34	31	.....	(*)	18	19	*14
11	30	6	85	144	30	*13	30	25	22	18	29	16
12	36	*65	*300	160	50	28	27	14	19	18	*29	22
13	54	35	87	585	25	20	19	(*)	19	16	29	64
14	41	35	85	350	*9	19	22	25	8	7	24	28
15	*185	29	82	253	47	30	13	22	5	*13	24	22
16	57	29	71	*228	36	34	*30	18	15	18	21	26
17	68	33	50	122	52	38	23	5	(*)	15	24	*35
18	48	37	43	118	47	*7	27	18	19	16	21	34
19	30	*13	*447	119	51	30	27	5	19	18	*24	22
20	36	48	80	98	52	30	19	(*)	13	14	26	42
21	42	58	65	90	*12	23	11	22	14	12	26	54
22	*13	409	87	53	52	21	3	19	21	*18	26	26
23	48	100	90	*86	35	25	(*)	16	10	24	25	22
24	52	62	90	59	36	23	27	5	(*)	18	21	*30
25	44	50	55	80	29	*23	30	15	22	22	18	25
26	44	*65	*95	76	33	26	22	10	15	19	*22	22
27	42	173	69	58	20	20	27	(*)	22	15	24	38
28	14	62	79	54	*40	19	11	22	13	7	24	16
29	*59	.....	76	43	42	20	14	30	18	*19	21	19
30	36	.....	71	*50	42	21	(*)	18	10	32	21	13
31	30	.....	78	.....	43	.....	27	5	.....	26	.....	*14
Mean	58	58	111	127	38	23	20	16	14	17	26	29

* Sunday.

Daily discharge, in second-feet, of Sauquoit Creek at New York Mills, New York, for 1900.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.
1	86	87	69	*207	81	42	*5	23	27
2	46	77	81	337	81	20	38	23	*5
3	46	97	35	267	81	*29	38	34	5
4	40	*45	*21	135	87	42	5	10	23
5	40	69	81	135	30	42	38	*5	23
6	17	72	74	91	*43	35	86	30	17
7	*25	67	89	269	87	29	34	23	20
8	50	147	89	*207	81	40	*9	23	26
9	36	495	86	103	81	24	65	20	*5
10	42	65	38	87	74	*17	65	17	23
11	40	*67	*17	87	77	42	28	15	37
12	46	172	74	113	41	46	32	*5	5
13	18	1,413	74	95	*55	41	32	23	5
14	*9	130	68	87	87	25	22	20	10
15	50	92	68	*135	65	32	*5	17	17
16	43	68	63	85	26	24	42	20	*5
17	50	37	42	207	35	*5	35	20	13
18	50	*55	*13	297	35	29	35	37	17
19	306	74	68	170	24	41	28	*5	17
20	272	68	371	95	*29	38	32	30	17
21	*117	68	95	193	35	38	33	18	17
22	92	379	93	*295	35	25	*5	20	13
23	97	392	93	95	35	27	38	17	*9
24	115	34	55	103	35	*5	38	17	13
25	95	*13	*67	91	35	32	48	18	17
26	65	80	89	87	14	38	42	*5	13
27	22	47	68	87	*33	25	32	30	17
28	*68	74	87	49	28	32	22	37	13
29	83	.....	86	*55	22	57	*5	35	13
30	91	.....	199	91	5	33	38	35	*5
31	63	.....	165	.....	35	.....	35	37	.....
Mean	72	146	84	138	49	32	32	22	15

* Sunday.

WEST CANADA CREEK AT TWIN ROCK BRIDGE, NEW YORK.

Twin Rock Bridge crosses West Canada Creek 2 miles above Hinckley, at practically the point of emergence of the stream from the Adirondacks. The bridge is of iron, has two spans, and is 167.5 feet long between abutments. The stream bed is of gravel and rock, and the conditions are unusually favorable for a current-meter station. A gage board was set and a record commenced on September 7, 1900. The gage is read at 7 a. m. and at 6 p. m. each day, and the average of the two daily readings is given in the appended table. A current-meter measurement made on September 7 showed the discharge to be 182 second-feet. The gage reading was 0.37 foot.

The record at Twin Rock Bridge is kept by the Utica Electric Light and Power Company. This company is erecting a power plant at Trenton Falls, 4 miles farther downstream. A concrete dam has been constructed, which will give a head of 265 feet on the turbines, which are of special design. It is the intention, after the plant is completed, to keep a continuous record of the amount of water used by the turbines and of the flow over the spillways.

The drainage areas at the different gaging stations are as follows:

Drainage areas of West Canada Creek.

	Square miles.
At mouth	569
At Middleville	519
At Trenton Falls	375
At Twin Rock Bridge	252

Daily gage height, in feet, of West Canada Creek at Twin Rock Bridge, New York, for 1900.

Day.	Sept.	Oct.	Nov.	Dec.	Day.	Sept.	Oct.	Nov.	Dec.
1		0.85	0.90	1.60	17	0.45	1.05	1.50	1.50
2		.75	1.60	1.85	18	.30	1.00	1.80	1.50
3		.65	1.10	2.00	19	.30	.95	5.50	1.50
4		.70	1.05	1.90	20	1.65	1.15	5.30	1.50
5		.60	1.15	1.75	21	2.90	.62	5.30	1.50
6		.55	1.70	1.55	22	2.35	.60	4.45	1.50
7		.65	4.55	1.40	23	1.50	2.30	3.75	1.75
8		.85	4.25	1.40	24	.95	2.45	2.00	1.70
9	0.35	.75	3.05	1.50	25	1.10	1.75	6.25	1.65
10	.32	.90	1.45	1.50	26	.90	1.70	6.20	1.60
11	.35	.85	2.05	1.50	27	.95	2.55	4.40	1.55
12	.32	.70	1.90	1.50	28	1.00	1.95	3.55	1.50
13	.35	.60	1.55	1.50	29	.85	1.15	1.90	1.50
14	.35	1.25	1.60	1.50	30	.90	1.20	1.70	
15	.38	1.30	1.50	1.50	31		1.50		
16	.78	1.10	1.50	1.50					

WEST CANADA CREEK AT MIDDLEVILLE, NEW YORK.

A description of this station will be found in Water-Supply Paper No. 35, page 49. In the past the principal element of uncertainty with regard to this record was considered to be the leakage of the dam, etc., which had been taken at 50 second-feet. Current-meter measurements were made on September 10, 1900, to determine the leakage of the dam and the low-water flow of the stream at this station, with the following results:

	Second-feet.
Highway bridge below dam, measured flow in main stream channel	113
Measured flow in hydraulic canal	132
Total flow by current-meter measurements	245

The calculated flow from the gage record is as follows:

	Second-feet.
Flow over dam, gage reading 0.67 foot	60
Leakage previously estimated	50
Total flow in main channel	110
Calculated diversion to water wheels	131
Total estimated flow	241

Unfortunately no current-meter measurements to check the calculated flow during high water have been made. The highest water observed was in August, 1898, when it reached a depth of 5.5 feet on the crest of the Middleville dam. The discharge at that time can not accurately be estimated, as a part of the water flowed around the end of the dam and passed overland to the stream channel below. High-water marks at the Newport dam indicate that the spring freshet dis-

charge has been as great as 22,000 second-feet, or 46.6 second-feet per square mile, from a drainage area of 472 square miles. The lowest water in this stream occurs on Sundays, when the flow is held back, as pond storage, by dams above Middleville. Aside from this, the most notable low-water period was September 2 to 12, inclusive, 1899, the mean flow at Middleville for eleven days being 183 second-feet per square mile.

Daily discharge, in second-feet, of West Canada Creek at Middleville, New York, for 1898.

[Drainage area, 519 square miles.]

Day.	Oct.	Nov.	Dec.	Day.	Oct.	Nov.	Dec.
1	.....	777	650	18	968	752	*1,349
2	.....	670	580	19	783	800	1,374
3	.....	584	580	20	689	*970	1,469
4	.....	595	*539	21	737	904	1,288
5	.....	450	589	22	981	801	1,614
6	.....	*550	651	23	*1,894	990	2,072
7	1,662	911	588	24	1,758	983	2,068
8	407	626	465	25	1,299	775	*1,739
9	*404	711	483	26	1,055	602	1,425
10	398	1,679	624	27	2,436	*740	1,181
11	447	4,240	*519	28	2,554	1,224	892
12	461	4,163	531	29	1,580	679	711
13	491	*2,410	552	30	*1,129	713	1,082
14	569	1,318	742	31	979	.....	1,492
15	2,152	1,071	1,123				
16	*1,894	788	1,362	Mean	1,161	1,110	1,024
17	1,263	816	1,413				

* Sunday.

Daily discharge, in second-feet, of West Canada Creek at Middleville, New York, for 1899.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	*1,216	1,135	1,563	821	2,979	1,114	198	273	257	*480	1,281	330
2	1,241	1,205	1,354	*523	4,760	763	*140	292	191	510	2,867	580
3	1,378	1,335	1,208	620	2,696	593	200	345	*145	470	1,744	*710
4	1,376	1,340	1,191	622	2,735	*550	116	316	145	436	1,199	710
5	3,445	*1,292	*1,348	680	1,926	531	240	237	210	337	*1,040	518
6	3,500	1,203	1,635	680	1,455	480	240	*155	205	253	831	540
7	1,858	1,101	1,835	748	*1,238	294	256	286	195	288	469	1,750
8	*1,315	1,172	1,207	1,697	1,187	314	204	262	219	*190	542	1,670
9	1,173	937	942	*1,503	2,170	258	*675	256	169	232	482	840
10	782	1,535	812	1,601	1,422	182	993	242	*145	220	390	*700
11	462	1,350	751	1,588	1,073	*130	807	246	188	234	420	640
12	786	*1,690	*1,340	2,230	1,189	283	443	214	201	210	*410	3,150
13	1,119	1,418	1,724	2,465	1,003	297	445	*125	221	239	342	4,710
14	1,132	1,227	1,632	3,228	*995	248	343	254	205	219	313	2,530
15	*1,615	1,666	1,181	3,582	1,196	753	236	216	204	*140	372	740
16	1,760	2,640	1,071	*3,513	888	839	*430	253	189	241	343	840
17	1,771	2,500	814	3,549	1,143	499	600	239	*145	238	363	*640
18	1,102	2,111	944	3,751	1,064	*320	560	235	197	235	372	1,060
19	621	*2,410	*1,060	4,477	1,183	339	479	174	208	217	*220	1,880
20	519	1,562	1,175	5,717	1,633	331	345	*110	213	152	337	3,520
21	563	1,816	1,555	5,564	*1,240	373	327	221	239	261	310	2,530
22	*535	1,966	1,412	5,381	1,274	346	225	245	238	*190	363	1,400
23	568	2,044	1,179	*6,083	996	263	*140	257	204	245	293	990
24	616	1,764	1,180	5,956	884	203	241	229	*145	225	273	*1,150
25	701	1,660	1,203	5,904	844	*155	241	254	209	238	273	900
26	763	*1,490	*1,313	6,994	801	283	241	183	476	231	*249	840
27	640	1,564	764	6,208	728	273	244	*110	433	267	312	580
28	626	1,444	795	5,779	*810	280	239	255	248	224	292	540
29	*605	.....	762	4,969	1,255	308	171	247	213	*965	310	640
30	818	.....	685	*4,513	1,179	275	*115	262	386	1,040	310	540
31	1,010	.....	758	.....	1,191	.....	234	289	.....	629	.....	*860
Mean	1,150	1,594	1,176	3,365	1,456	397	324	235	221	324	577	1,259

* Sunday.

Daily discharge, in second-feet, of West Canada Creek at Middleville, New York, for 1900.

Day.	Jan.	May.a	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	824	.....	415	*320	362	457	470	450	1,517
2	722	.....	358	338	450	*320	469	320	*1,222
3	984	.....	*530	381	450	459	426	285	1,025
4	1,001	.....	490	330	315	453	401	*270	1,175
5	1,219	.....	524	299	*190	464	336	270	1,325
6	1,227	.....	522	289	249	494	219	340	1,335
7	*1,240	.....	491	358	235	534	*198	360	1,004
8	1,244	.....	557	*500	206	421	369	880	1,030
9	1,156	.....	676	523	205	*280	467	1,010	*818
10	1,106	1,667	*480	448	168	401	404	1,010	677
11	972	1,643	536	454	200	393	339	*700	573
12	937	1,228	495	538	*363	324	327	580	483
13	930	*1,429	420	442	437	278	330	430	483
14	*965	1,087	412	350	971	270	*280	430	483
15	1,011	1,281	368	*320	1,021	176	576	430	399
16	1,051	1,069	341	439	1,516	*136	726	340	*390
17	1,068	1,127	*284	424	541	227	646	340	413
18	1,442	1,268	332	517	490	266	466	*410	413
19	2,440	1,205	389	523	*370	282	466	580	465
20	1,485	*1,362	334	487	415	291	324	1,430	502
21	*2,680	959	281	547	344	321	*170	2,900	564
22	2,403	744	316	*530	327	818	290	2,630	539
23	2,006	637	298	488	269	*947	299	2,340	*500
24	2,874	595	*284	349	258	940	778	1,900	812
25	2,396	544	273	511	398	608	958	*3,120	942
26	2,093	437	219	636	*320	574	727	5,920	898
27	2,394	*390	306	567	506	464	461	5,670	900
28	*730	466	326	445	551	364	*460	2,900	900
29	870	423	520	*373	674	286	464	1,980	900
30	780	380	431	481	599	*320	454	1,665	*1,020
31	530	413	.....	386	534	.....	594	.....	955
Mean	1,366	924	406	451	463	419	448	1,536	800

a No record for February, March, and April.
* Sunday.

MOHAWK RIVER AT LITTLE FALLS, NEW YORK.

This gaging station, which has been described in Water-Supply Paper No. 35, page 51, is located at the lower (Gilbert's) dam at Little Falls. Current-meter measurements have been made to check the calculated flows, with very satisfactory results. They are as follows:

		Second-feet.
October 20 to 21, 1898, at suspension bridge 2 miles below Little Falls, W. D. Lockwood, hydrographer:		
Total flow by current meter		1,758
Computed flow (mean of two days)		1,733
May 1, 1900, at suspension bridge 2 miles below Little Falls, R. E. Horton, hydrographer:		
Total flow by current meter		4,773
Computed flow over dam	4,060	
Computed diversion to Gilbert's mill	183	
Computed diversion to paper mill	556	
Total computed flow		4,799
May 23, 1900, at Astronga Bridge, Little Falls, R. E. Horton, hydrographer:		
Total flow by current meter		1,567
Computed flow over dam	950	
Computed diversion to Gilbert's mill	176	
Computed diversion to paper mill	405	
Total computed flow		1,531

Diversion to paper mill, September 19, 1900:

Metered flow in headrace	302
Computed flow through turbines, etc	288

There are three dams at Little Falls. The two lower are used for water-power development; the upper one is a State dam, diverting water for the supply of Erie Canal. The gage record kept at the lower dam shows the amount of water flowing downstream from Little Falls, but does not include the diversion at the State dam above the gaging station, and hence does not represent the total yield or inflow from the tributary drainage area of 1,306 square miles.

Current-meter measurements were made in the feeder channel below the State dam, as follows:

Current-meter measurements in feeder channel below State dam.

Date.	Measured diversion.
1900.	*Second-feet.*
May 23	143
September 19	179

Adding the foregoing amounts to the mean daily flow at Gilbert's dam for the same dates, we obtain the following:

Volumes of inflow and outflow at Little Falls diversion.

Date.	Total inflow at Little Falls.	Outflow in main channel.
1900.	*Second-feet.*	*Second-feet.*
May 23	1,596	1,453
September 19	693	514

Water is again diverted to the canal at Fivemile dam, below Little Falls. On November 3, 1900, the measured flow in this feeder, at Lansing's farm bridge, was 236 second-feet.

High water occurred at Little Falls as follows:

High water at Little Falls.

Date.	Depth of water on crest of dam.	Volume.	
	Feet.	*Second-feet.*	*Sec.-ft. per sq. mile.*
April 15, 1899	7.33	13,000	10.0
April 20, 1900	8.21	15,240	11.7

The most notable low-water period was August 3 to August 10, inclusive, 1899, the mean flow for eight days being only 120 second-feet, or 0.07 second-foot per square mile.

Daily discharge, in second-feet, of Mohawk River at Little Falls, New York, for 1898.

[Drainage area, 1,306 square miles.]

Day.	Sept.	Oct.	Nov.	Dec.	Day.	Sept.	Oct.	Nov.	Dec.
1	.....	1,121	2,261	1,499	18	.....	2,265	2,220	*1,173
2	.....	*916	1,880	1,547	19	.....	1,699	2,399	1,499
3	.....	849	1,790	1,451	20	.....	1,745	*2,878	1,499
4	.....	895	1,560	*1,173	21	.....	1,722	2,771	1,854
5	.....	2,083	1,345	1,553	22	.....	2,646	2,361	2,909
6	.....	1,627	*1,501	1,763	23	1,125	*4,096	2,794	4,734
7	.....	1,558	1,946	1,639	24	3,484	4,339	2,694	5,279
8	.....	1,179	2,040	1,269	25	*4,093	3,240	2,538	*4,708
9	.....	*916	1,700	969	26	2,718	2,574	1,950	3,759
10	.....	900	4,420	1,462	27	2,445	6,290	*1,473	2,309
11	.....	867	9,433	*1,508	28	2,220	6,188	1,137	1,899
12	.....	915	7,925	1,709	29	1,615	5,121	1,412	1,499
13	.....	1,028	*6,728	1,409	30	1,320	*3,246	1,562	1,929
14	.....	1,040	5,245	1,259	31	.....	2,628	.....	3,979
15	.....	5,490	3,520	1,309					
16	.....	*5,026	2,891	1,259	Mean	2,378	2,495	2,891	2,096
17	.....	3,061	2,370	1,434					

Sunday.

Daily discharge, in second-feet, of Mohawk River at Little Falls, New York, for 1899.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July	Aug.	Sept.	Oct.	Nov.	Dec.
1	*2,854	941	3,258	3,041	6,360	1,957	569	506	278	*704	2,829	2,096
2	1,796	906	3,154	*2,519	6,708	1,610	*498	506	271	651	6,600	1,709
3	1,846	861	2,304	2,329	5,619	1,321	359	135	*30	651	4,738	*1,962
4	1,936	884	2,298	2,536	3,925	*1,075	424	136	396	491	3,081	2,175
5	6,821	*646	*3,408	2,387	2,905	1,466	490	166	545	534	*3,262	2,097
6	7,440	1,063	5,544	4,297	2,339	1,216	663	*000	382	399	2,847	1,668
7	5,886	983	5,544	4,637	*1,560	1,016	627	133	272	240	1,908	1,469
8	*4,269	983	5,154	7,212	1,801	1,002	651	131	275	*160	1,718	1,588
9	4,377	846	4,534	*6,759	1,571	902	*1,660	131	211	357	1,628	1,630
10	2,529	803	3,054	7,079	1,470	764	2,593	130	*7	297	1,451	*1,016
11	1,467	871	2,572	6,729	1,469	*1,176	1,800	316	273	274	717	2,779
12	1,378	*789	*3,967	7,979	1,846	660	1,320	383	270	305	*1,112	6,704
13	1,298	983	6,614	9,644	2,273	660	1,222	*000	270	274	1,526	10,996
14	1,992	903	6,529	12,484	*1,920	685	941	329	211	258	1,448	8,435
15	*4,104	1,222	6,304	12,502	2,212	900	884	352	230	*196	1,150	7,367
16	4,848	1,333	6,354	*12,649	1,746	1,800	*643	351	184	354	1,069	3,500
17	4,703	748	2,129	11,339	1,896	1,800	1,315	244	*55	339	1,120	*1,328
18	4,279	1,326	2,254	10,716	2,492	*965	1,211	251	278	344	1,037	1,682
19	1,509	*669	*3,195	10,869	2,843	568	1,086	216	267	339	*984	4,812
20	1,608	1,248	3,658	11,638	3,742	1,010	1,008	*65	281	355	1,070	8,540
21	1,687	1,523	2,704	12,159	*3,650	1,105	767	158	220	331	1,120	7,216
22	*1,794	2,598	3,429	10,878	3,143	985	95	223	249	*142	1,145	5,822
23	1,885	4,506	3,494	*10,409	2,172	866	*363	235	129	350	1,086	5,147
24	1,859	4,001	3,694	10,920	1,696	745	496	203	*55	401	987	*1,753
25	2,284	3,664	3,824	9,870	1,470	*851	496	203	238	392	900	2,186
26	2,060	*1,456	*3,258	8,990	1,309	759	516	246	225	369	*829	1,277
27	1,813	2,376	2,829	9,180	1,290	634	496	*136	556	387	822	1,571
28	1,175	3,201	2,345	7,880	*1,680	585	510	348	920	314	969	1,641
29	*1,574	.....	2,514	7,139	2,323	560	460	188	642	*1,398	969	1,380
30	1,154	.....	3,029	*6,299	3,827	535	*50	219	723	2,293	886	1,049
31	1,126	.....	3,029	.....	2,903	.....	697	277	.....	1,882	.....	*932
Mean	2,753	1,510	3,757	8,102	2,651	1,014	803	223	298	509	1,699	3,360

* Sunday.

Daily discharge, in second-feet, of Mohawk River at Little Falls, New York, for 1900.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	595	1,594	1,454	*5,785	4,673	645	*395	481	629	667	1,289	4,845
2	961	1,372	1,496	8,497	4,510	761	1,053	509	*545	849	1,289	*4,027
3	1,040	1,251	1,539	10,124	4,362	*1,119	1,804	515	455	881	1,197	3,200
4	1,040	*421	*1,001	9,162	4,014	1,346	311	359	503	499	*1,264	3,109
5	608	1,457	2,228	8,114	3,774	1,174	644	*93	503	491	885	6,316
6	578	1,682	2,013	7,936	*2,443	1,014	505	284	503	493	920	5,504
7	*180	1,594	1,967	9,998	1,658	760	788	274	629	*282	982	5,727
8	1,592	1,814	2,125	*10,065	1,698	1,214	*625	195	228	449	3,350	5,620
9	1,966	4,712	2,228	8,587	2,304	1,643	1,048	374	*326	665	5,335	*4,725
10	1,921	5,053	2,250	6,660	2,591	*1,283	876	452	294	747	3,837	2,579
11	1,772	*3,449	*1,889	5,053	2,595	1,134	754	153	326	747	*2,678	2,562
12	1,726	5,305	2,013	4,532	2,390	1,134	754	*181	200	499	1,901	2,388
13	1,576	10,192	1,818	4,304	*2,132	864	694	365	483	598	1,738	2,388
14	*1,731	11,128	1,875	4,202	1,462	864	666	2,041	515	*459	1,600	2,178
15	1,636	11,642	1,818	*4,607	1,859	896	*545	2,370	377	691	1,598	1,586
16	1,496	8,292	1,726	5,672	1,732	801	668	1,434	*461	1,231	1,368	*1,866
17	1,455	8,961	1,696	7,261	1,927	*555	724	1,064	515	1,062	1,207	1,630
18	1,455	*3,671	*1,380	11,128	1,778	702	826	874	713	983	*1,336	2,230
19	1,592	3,678	1,496	13,542	1,882	599	754	*745	501	881	1,998	1,825
20	4,302	2,334	1,967	15,242	*1,745	561	794	651	492	785	3,929	2,361
21	*7,291	1,968	3,345	15,032	1,732	650	1,170	494	599	*622	6,057	2,180
22	7,194	2,386	3,740	*11,275	1,637	579	*1,430	494	1,777	564	5,353	1,981
23	7,736	4,303	3,740	10,624	1,453	437	1,068	499	*1,380	643	4,515	*1,828
24	6,150	3,614	4,135	9,437	1,453	*72	696	475	953	1,114	3,417	3,509
25	4,740	*1,681	*3,720	8,497	1,165	474	1,424	527	1,034	2,137	*3,094	5,291
26	4,670	1,592	3,405	7,282	1,085	390	2,109	*795	915	1,884	12,063	5,177
27	4,242	1,456	3,030	6,921	*1,014	474	1,898	689	819	1,470	14,551	4,143
28	*1,528	1,541	3,030	5,490	751	474	1,384	657	914	*1,356	13,520	3,007
29	2,176	.....	3,150	*4,559	751	674	*1,262	1,255	652	1,632	8,615	2,606
30	1,772	.....	4,233	4,672	723	740	874	959	*668	1,822	4,734	*1,909
31	1,496	.....	5,024	.....	723	.....	696	721	.....	1,192	.....	2,159
Mean	2,523	3,862	2,469	8,142	2,063	801	943	694	630	899	3,854	3,240

*Sunday.

EAST CANADA CREEK AT DOLGEVILLE, NEW YORK.

A description of this station, together with the estimated daily discharge as originally computed, June to December, 1899, will be found in Water-Supply Paper No. 35, page 52. A new rating table for the dam has been prepared, using coefficients of discharge derived from Freeman's experiments on a model of the round-crested portion of the Croton dam, which apparently corresponds closely with the ogee section of the Dolgeville dam, as regards friction on the crest, vertical contraction of the nappe, and siphonage.[1]

In computing the record here given, the new rating table has been used, beginning June 1, 1899. The flow through the turbines has also been computed from current-meter measurements made in the tailrace of the electric power plant, instead of from the observed head and the gate openings of the water wheels, as formerly. The effect of these changes has been to slightly increase the extreme high-water and low-water flows, the estimated flow for mean stages remaining substantially the same.

[1] See Report on New York's Water Supply, made to Bird S. Coler, comptroller (1900), by John R. Freeman, p. 137.

Current-meter measurements were made from the bridge across the tailrace below the power plant, as follows:

Current-meter measurements of flow in tailrace at Dolgeville electric-light and power dam.

Date.	Flow in tailrace.	Gate opening of 36-inch wheels.	Wheel.
1900.	*Second-feet.*	*Per cent.*	
May 22	84	0.50	No. 2.
July 27	76	.38	No. 2.
July 29	63	.28	No. 1.
August 9	80	.38	No. 1.

Only one of the two 36-inch Victor turbines was running in each instance, together with the 15-inch exciter wheel. The depths of gate openings of the 36-inch wheels are shown in the foregoing table. The 15-inch exciter wheel ran at thirteen-hundredths gate in each case. The head on the wheels was 72 feet. Observations of the wheel-gate openings were taken at the beginning and end of each test, and the average is given in the table.

The results of the current-meter measurements of the total flow of the stream have been given in the general table on page 39.

The relation between the metered and calculated flows on different days is shown in the following tables, from gage readings taken at the time the measurements were made:

Current-meter measurements of East Canada Creek.

Date.	Flow in tailrace.	Flow over dam.	Total flow.
1900.	*Second-feet.*	*Second-feet.*	*Second-feet.*
May 22	84	328	412
July 27	76	376	452
August 7	80	*a* 28	108

a Measured in stream channel above point of confluence with tailrace.

Calculated flow of East Canada Creek.

Date.	Crest gage reading.	Flow over dam.	Flow in tailrace.	Total flow.
1900.	*Feet.*	*Second-feet.*	*Second-feet.*	*Second-feet.*
May 22	0.69	282	84	366
July 27	.79	362	84	446
August 7	.20	29	78	107

In the first two cases the total flow was measured at the Dolgeville bridge above the dam, and the difference between the observed and calculated flows in the first instance is probably due to pond storage.

The accompanying tables of mean daily flow show the amount of water passing down the stream from the dam each day, with the exception of Sundays, for which the flow has been taken as a mean between

that of the preceding and following days. Dams on this stream are not numerous, nor is there extensive pond storage, so that the tables may be taken as a fair representation of the natural regimen of flow.

The highest water observed was on April 19, 1900, when a depth of 4.5 feet on the crest of the dam was reached, corresponding to a flow of 5,750 second-feet, or 22.6 second-feet per square mile. The most notable low-water period was September 13 to 16, inclusive, 1899, when the average volume of flow was 67 second-feet, or 0.3 second-foot per square mile.

Daily discharge, in second-feet, of East Canada Creek at Dolgeville, New York, for 1898.

[Drainage area, 256 square miles.]

Day.	Sept.	Oct.	Nov.	Dec.	Day.	Sept.	Oct.	Nov.	Dec.
1		275	508	372	18		465	540	(a)
2		*330	478	372	19		447	540	(a)
3		275	443	372	20		477	*630	400
4		270	363	*372	21		452	570	650
5		310	343	372	22		637	535	775
6		330	*395	372	23	485	*1,323	500	1,275
7		290	398	(a)	24	1,180	1,082	465	975
8		280	378	(a)	25	943	832	440	*700
9		*213	398	(a)	26	680	762	370	600
10		252	648	(a)	27	625	1,222	*305	600
11		242	333	(a)	28	465	1,082	235	500
12		267	1,937	(a)	29	370	877	235	400
13		287	*1,620	(a)	30	355	*635	279	450
14		322	1,360	(a)	31		602		600
15		1,422	1,110	(a)					
16		*1,073	850	(a)	Mean	638	581	689	564
17		690	590	(a)					

*Sunday. a Ice on crest of dam.

Daily discharge, in second-feet, of East Canada Creek at Dolgeville, New York, for 1899.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	*486	472	542	462	1,701	384	116	118	109	*135	1,046	184
2	486	462	447	*359	1,401	294	*82	192	145	118	1,674	292
3	530	447	372	355	1,301	218	112	210	*145	106	874	*335
4	530	492	422	412	1,261	*212	79	139	78	106	757	388
5	1,320	*370	*833	452	1,006	206	106	99	78	94	*648	282
6	1,390	402	852	512	771	200	103	*81	71	89	540	234
7	1,135	397	787	642	*474	194	126	118	68	89	404	152
8	*922	397	682	922	421	234	215	112	67	*94	372	152
9	922	397	542	*713	371	194	*324	100	67	92	340	132
10	722	372	472	822	371	194	394	106	*67	89	322	*195
11	622	372	447	952	421	*183	270	126	67	77	282	258
12	542	*348	*550	1,217	401	194	194	143	74	75	*258	2,000
13	502	410	742	1,642	431	194	194	*123	67	74	234	3,029
14	562	462	772	2,152	*534	206	194	103	67	74	234	1,530
15	*342	502	722	2,972	601	264	175	83	67	*74	234	1,094
16	1,187	502	602	*2,077	541	249	*304	78	67	74	288	914
17	1,167	397	562	2,512	441	175	374	104	*67	77	258	*733
18	1,942	402	472	2,682	601	*134	264	78	67	118	234	1,325
19	822	*370	*490	3,532	811	239	175	76	67	100	*234	1,947
20	742	447	542	4,182	741	161	126	*75	74	103	234	676
21	642	477	572	4,472	*674	274	119	74	74	100	208	1,422
22	*542	590	602	3,472	601	206	100	74	74	*90	292	874
23	572	992	602	*3,791	541	174	*112	67	74	81	292	757
24	652	742	702	3,972	471	133	106	74	*74	100	184	*648
25	672	512	672	3,992	421	*112	110	94	74	94	152	542
26	672	*460	*520	4,132	391	152	135	74	192	94	*148	436
27	642	742	447	3,782	371	152	111	*44	192	89	143	330
28	602	742	492	3,592	*324	144	135	74	139	89	143	282
29	*542	.....	472	3,017	391	119	115	74	126	*266	134	282
30	522	.....	492	*2,141	373	119	*101	74	152	372	134	254
31	472	.....	472	.....	421	.....	87	74	.....	246	.....	*259
Mean	816	439	519	1,978	633	196	166	97	92	112	377	706

*Sunday.

Daily discharge, in second-feet, of East Canada Creek at Dolgeville, New York, for 1900.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	264	288	252	*631	1,476	(a)	*130	153	132	120	(a)	(a)
2	264	288	240	794	1,188	(a)	112	126	*151	185	242	*(a)
3	288	288	252	914	800	(a)	112	110	129	145	218	(a)
4	288	*288	*227	914	610	(a)	115	111	190	144	*194	(a)
5	300	288	240	914	474	591	118	*98	167	110	242	1,192
6	346	266	192	1,626	*410	394	152	82	149	116	218	844
7	*346	264	152	2,321	346	372	184	89	157	*100	242	614
8	410	288	192	*2,714	474	608	*193	98	118	94	632	614
9	410	800	258	1,374	610	827	200	108	*118	99	722	*577
10	372	718	340	1,094	800	*643	182	98	73	94	582	614
11	362	*718	*321	1,002	610	460	167	87	152	85	*482	279
12	318	718	306	1,012	546	406	145	*111	142	84	398	279
13	300	3,759	270	1,020	*454	376	132	192	142	84	(a)	224
14	*258	4,320	234	1,020	362	394	124	396	136	*81	(a)	224
15	264	2,467	234	*1,197	342	376	*137	274	128	238	(a)	210
16	252	1,632	192	1,374	342	335	150	250	*93	205	(a)	*82
17	240	1,232	152	1,926	288	*304	162	185	117	199	(a)	63
18	682	*939	*173	4,626	288	274	202	167	104	186	*(a)	210
19	546	646	262	5,335	288	256	190	*133	95	180	(a)	215
20	960	562	306	4,917	*264	256	158	92	73	185	1,005	345
21	*1,704	518	340	4,842	240	250	310	110	196	*158	1,197	215
22	1,680	540	340	*4,368	288	246	*252	111	184	153	1,148	215
23	1,144	709	340	3,895	288	326	194	142	*130	171	1,016	*342
24	1,012	562	340	3,095	288	*300	374	117	154	582	713	310
25	820	*470	*340	2,840	276	274	686	117	99	510	*1,146	438
26	709	378	306	2,355	264	294	767	*77	(a)	322	3,802	345
27	604	346	282	2,243	(a)	294	422	92	(a)	350	3,164	345
28	*508	312	282	1,586	(a)	300	276	223	(a)	*306	1,689	317
29	410	------	306	*1,506	(a)	300	*219	136	(a)	292	1,128	317
30	346	------	420	1,207	(a)	149	162	181	*(a)	268	929	*252
31	346	------	468	------	(a)	------	110	151	------	(a)	------	263
Mean	531	879	276	2,086	486	370	221	144	133	195	957	268

a No record. * Sunday.

CAYADUTTA CREEK NEAR JOHNSTOWN, NEW YORK.

A description of this station will be found in Water-Supply Paper No. 35, page 53. The record is kept at the dam of the Johnstown Electric Light and Power Company, 1 mile below Johnstown. Since the establishment of the station standard sharp-crested gaging weirs have been erected by Prof. Olin H. Landreth, C. E. One of these weirs has been placed across the main stream above the head of slack water from the dam. A second weir has been placed in the tailrace below the power house.

During the summer the water does not ordinarily flow over the dam, which is practically tight, but the entire flow passes through the turbines. A series of gagings at the tailrace weir has been made in order to determine the discharging capacity of the water wheels when running under different conditions.

Dams are located at frequent intervals along the stream, and during the dry season the amount of flow from one to another is largely controlled by the water wheels.

Daily discharge, in second-feet, of Cayadutta Creek near Johnstown, New York, for 1898.

[Drainage area, 40 square miles.]

Day.	Oct.	Nov.	Dec.	Day.	Oct.	Nov.	Dec.
1	24	41	39	18	47	57	*27
2	*10	36	48	19	46	97	33
3	22	38	46	20	39	*67	35
4	25	32	*51	21	47	67	45
5	119	39	23	22	55	50	58
6	246	*12	51	23	*105	54	98
7	67	32	50	24	56	63	67
8	55	35	39	25	48	55	*60
9	*14	179	38	26	36	42	75
10	37	734	38	27	58	*24	42
11	23	429	*25	28	55	46	31
12	27	110	46	29	138	59	34
13	33	*64	37	30	*13	42	50
14	38	68	36	31	41	.....	44
15	290	55	33				
16	*117	49	32	Mean	64	91	44
17	66	57	34				

*Sunday.

Daily discharge, in second-feet, of Cayadutta Creek near Johnstown, New York, for 1899.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	*20	27	66	72	57	30	21	20	16	*12	33	19
2	39	25	48	*70	66	28	*15	22	20	24	31	.....
3	31	29	36	83	36	29	24	32	*12	24	28	*14
4	42	26	40	132	32	*17	17	22	19	25	48	37
5	71	*19	*111	252	28	29	20	21	28	17	*34	27
6	69	29	282	263	27	28	20	*11	16	15	26	23
7	40	23	122	313	*12	18	19	19	23	19	28	23
8	*64	19	71	508	29	30	34	20	22	*9	26	23
9	42	18	72	*320	26	21	*16	20	17	18	22	23
10	29	18	67	413	25	19	27	20	*7	20	27	*13
11	24	20	53	579	27	*18	25	17	16	24	26	27
12	27	*14	*123	415	47	30	18	23	19	23	*20	238
13	26	19	155	811	29	25	18	*8	20	13	28	207
14	43	21	105	751	*31	24	16	20	17	20	28	145
15	*125	24	84	802	25	25	18	20	19	*17	30	55
16	51	25	46	*218	28	25	*13	20	20	24	20	40
17	43	26	38	190	34	25	23	20	*11	26	21	*44
18	35	27	35	196	33	*17	25	20	19	33	20	36
19	31	*15	*66	225	43	25	26	19	20	21	*6	51
20	33	32	67	168	47	37	20	*9	21	.....	27	85
21	28	33	66	135	*34	32	19	15	20	.....	27	62
22	*16	55	50	142	34	33	23	16	17	(*)	26	42
23	35	91	74	*123	26	32	*8	14	19	.....	25	26
24	40	39	59	41	31	32	18	16	*14	.....	24	*19
25	27	40	52	65	26	*18	22	16	20	21	26	56
26	32	*43	*54	66	25	29	19	14	32	18	*14	60
27	32	53	48	44	30	26	16	*11	36	18	30	25
28	24	48	54	52	*15	30	17	15	26	19	28	24
29	*13	.....	48	60	32	30	17	16	27	*16	28	20
30	33	.....	46	*81	13	22	*14	16	26	30	15	19
31	33	.....	48	.....	28	.....	25	16	.....	30	.....	*11
Mean	39	31	74	251	31	26	20	18	20	21	26	49

*Sunday.

Daily discharge, in second-feet, of Cayadutta Creek near Johnstown, New York, for 1900.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept
1	27	39	41	*250	38	36	*6	12	11
2	27	29	42	292	25	22	14	22	*7
3	25	22	44	339	25	*8	20	20	21
4	20	*20	*23	212	25	30	6	18	17
5	18	251	55	199	28	26	15	*7	17
6	20	43	41	200	*24	25	15	20	17
7	*24	42	50	293	30	23	15	20	17
8	60	148	45	*155	25	24	*7	20	21
9	39	235	45	102	81	33	31	22	*8
10	43	76	44	78	63	*13	19	18	21
11	37	*82	*39	77	30	29	18	14	20
12	34	116	66	91	24	29	20	*4	6
13	30	1,404	48	106	*19	16	23	27	7
14	*14	89	40	78	23	24	18	28	18
15	35	77	41	*91	26	29	*12	29	16
16	37	59	39	133	23	22	19	34	*9
17	29	46	41	200	24	*9	20	35	19
18	40	*23	*19	314	29	24	18	33	21
19	92	42	42	164	26	24	17	*10	19
20	705	41	45	102	*22	24	14	18	21
21	*306	39	52	76	25	22	19	21	21
22	101	44	52	*116	24	21	*7	20	21
23	83	107	111	92	19	17	15	21	*14
24	55	87	82	80	22	*10	18	22	32
25	71	*41	*52	71	23	17	17	22	32
26	50	60	46	64	26	19	26	*7	21
27	43	42	50	48	*19	14	25	20	21
28	*23	39	49	46	22	17	26	19	23
29	44	.....	106	*22	19	19	*13	20	23
30	36	.....	165	25	20	18	13	20	*10
31	31	.....	296	.....	22	.....	13	18	.....
Mean	71	119	62	137	27	21	17	20	18

* Sunday.

NOTE.--Records for October, November, and December are not available at present.

SCHOHARIE CREEK AT FORT HUNTER, NEW YORK.

Schoharie Creek rises on the western slope of the Catskill Mountains. In its lower stages it flows through a long, flat valley, in a stream bed covered with cobbles and gravel, over which the water finds its way in a thin sheet during the dry season.

The State dam at Fort Hunter is near the mouth of the stream, and high water from Mohawk River backs up to the toe of the dam. The condition existing at this station during the summer months has been described in connection with the Mohawk River gaging stations, page 42. A record has been kept of the elevation of the water surface in the pond above the dam, and also in the channel below the dam. The average difference, or head, is 5.25 feet, and it is nearly constant, except when water falls below the crest level above the dam.

The dam is of timber backed with gravel, and there are a number of leaks above the gravel line 2 feet below the crest. During the summer this leakage represents practically the total inflow from Schoharie Creek to the Mohawk. Current-meter measurements of the leakage were made below the Erie Canal aqueduct, at a point where the entire flow is concentrated in a narrow channel.

Current-meter measurements of leakage of Fort Hunter dam.

Date.	Hydrographer.	Leakage.
1900.		*Second-feet.*
June 21	R. E. Horton	30
July 18	E. D. Walker	38
August 22	do	44

In the computation the leakage of the dam has been assumed to be 35 second-feet.

In establishing this station the intention was to maintain a record of the height of the water above and below the head gates, at the entrance to the canal feeder, from which the effective head of the gate openings could be determined and the flow computed by the formula for submerged orifices. During the dry season the water falls below the lip of the gates and flows in an open channel, making this method inapplicable. In recomputing the record the diversion to the canal feeder was estimated from the following current-meter measurements:

Current-meter measurements of flow of water in feeder.

Date.	Hydrographer.	Measured flow.
1900.		*Second-feet.*
June 21	R. E. Horton	112
July 18	E. D. Walker	76
August 22	do	73

Inflow to the Erie Canal is controlled by gates at the lower end of the feeder channel, so that the flow in the feeder is not directly a function of the stage of the water. Owing to the uncertainty of the low-water measurements, this station was abandoned July 31.

The accompanying tables show the total outflow from the pond above the Fort Hunter dam. A table of the flow, as originally computed, allowing 315 second-feet for leakage, will be found in Water-Supply Paper No. 35, page 55.

The drainage areas tributary to Schoharie Creek at various gaging stations are as follows:

Drainage areas of Schoharie Creek.

	Square miles.
At mouth	947.0
At Erie Canal aqueduct	946.8
At Fort Hunter dam	946.7
At Mill Point bridge	934.0
At Schoharie Falls dam	930.0

Daily discharge, in second-feet, of Schoharie Creek at Fort Hunter, New York, for 1898.

[Drainage area, 947 square miles.]

Day.	Sept.	Oct.	Nov.	Dec.	Day.	Sept.	Oct.	Nov.	Dec.
1	.....	161	1,617	1,139	18	.....	791	1,517	1,539
2	.....	161	1,717	939	19	.....	861	2,417	859
3	.....	161	917	1,339	20	.....	791	3,967	659
4	.....	161	917	1,439	21	.....	791	2,842	814
5	.....	1,661	767	1,839	22	.....	791	2,067	1,639
6	.....	3,211	767	1,639	23	.....	1,061	1,967	2,639
7	.....	1,361	847	1,839	24	129	861	1,967	2,069
8	.....	791	767	1,839	25	129	861	1,517	3,269
9	.....	1,761	667	1,739	26	129	861	1,517	814
10	.....	791	5,067	1,239	27	129	1,361	1,217	1,539
11	.....	611	9,517	939	28	129	2,261	917	1,539
12	.....	611	5,767	1,639	29	129	1,911	767	939
13	.....	581	2,917	1,539	30	129	1,561	767	1,239
14	.....	1,661	2,967	1,639	31	.....	1,661	.....	3,864
15	.....	2,136	2,417	1,539					
16	.....	2,461	1,817	1,639	Mean	129	1,142	2,148	1,573
17	.....	711	1,517	1,339					

Daily discharge, in second-feet, of Schoharie Creek at Fort Hunter, New York, for 1899.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.
1	1,741	1,651	4,915	1,335	1,515	295	182	148	148	3,867	935
2	1,641	1,351	3,655	1,335	1,460	295	178	152	148	1,967	1,615
3	1,641	1,651	3,165	1,335	1,560	395	182	145	145	1,767	1,515
4	941	1,251	2,935	2,135	1,140	395	182	138	145	2,247	935
5	5,791	1,251	6,635	2,785	860	295	185	142	148	2,467	935
6	4,791	951	13,635	3,185	860	295	185	145	148	1,967	765
7	3,541	951	6,235	3,655	710	295	190	148	148	1,767	935
8	3,141	851	3,435	7,685	585	225	195	138	142	1,547	935
9	3,541	901	2,935	5,335	585	205	198	138	152	1,767	785
10	2,241	901	3,535	3,655	610	195	185	135	152	967	765
11	1,991	851	4,085	4,585	440	195	182	138	152	967	935
12	1,641	851	7,685	5,735	440	195	190	138	148	1,547	1,615
13	1,841	657	11,135	7,685	340	195	198	132	148	1,547	1,835
14	1,841	701	4,455	7,635	260	195	202	132	145	1,967	1,515
15	2,091	801	3,175	2,335	260	195	202	135	145	1,547	1,515
16	2,341	951	2,635	7,685	340	195	198	138	142	2,247	935
17	3,291	951	2,235	5,215	480	195	202	138	138	967	935
18	2,741	1,551	2,035	4,085	585	195	202	138	145	797	765
19	2,291	1,751	2,335	4,385	510	195	195	142	145	587	765
20	2,341	1,551	2,515	3,812	585	195	195	145	148	967	765
21	1,991	3,151	2,775	3,532	585	195	190	142	148	797	635
22	1,641	2,876	2,775	2,342	510	195	185	148	148	587	635
23	1,541	3,407	2,775	3,532	440	195	185	148	148	967	453
24	1,541	3,726	2,635	3,342	340	195	165	142	148	797	453
25	2,741	3,726	2,515	3,198	340	195	198	145	152	1,967	515
26	1,991	4,301	2,335	3,883	260	195	190	148	182	1,767	455
27	2,091	4,885	2,035	2,633	260	195	185	142	6,984	2,147	425
28	1,841	6,035	2,135	2,098	260	195	182	148	6,126	1,547	335
29	1,541	.....	1,435	2,298	340	195	190	145	5,740	1,967	335
30	1,641	.....	1,335	2,948	260	195	190	148	4,790	2,147	295
31	1,541	.....	1,435	.....	260	.....	190	145	.....	1,547	.....
Mean	2,307	1,944	3,792	4,100	579	228	187	142	916	1,603	873

NOTE.—No record for December.

Daily discharge, in second-feet, of Schoharie Creek at Fort Hunter, New York, for 1900.

Day.	Jan.	Mar.	Apr.	May.	June.	July.
1	735	3,465	2,915	835	360	111
2	735	7,335	2,775	985	155	102
3	575	7,515	3,335	835	245	102
4	575	6,785	3,635	985	245	111
5	435	5,585	3,335	1,208	284	90
6	315	2,485	3,335	490	239	76
7	435	3,335	3,635	412	212	76
8	315	3,335	2,635	409	242	76
9	485	3,035	2,495	315	515	135
10	435	2,740	2,135	519	276	140
11	315	3,335	2,015	472	360	143
12	365	3,035	2,015	390	305	140
13	485	2,775	2,275	300	305	135
14	575	2,355	2,915	293	257	127
15	435	2,255	2,915	312	260	123
16	575	2,015	3,635	327	260	118
17	575	1,885	4,735	319	235	118
18	735	1,755	6,035	312	220	111
19	2,785	1,755	4,935	330	157	111
20	4,595	1,755	4,595	1,530	153	111
21	8,235	3,035	5,355	1,550	147	106
22	4,595	2,135	6,015	1,020	143	72
23	3,465	1,755	6,615	500	139	122
24	3,145	2,495	5,095	412	139	130
25	1,325	2,275	4,035	409	139	144
26	735	2,635	3,335	315	105	135
27	735	2,495	2,915	315	113	127
28	485	2,775	2,635	250	122	127
29	575	3,035	1,885	322	122	118
30	485	3,035	1,325	555	122	118
31	435	3,035	.....	360	.....	111
Mean	1,313	3,137	3,530	561	219	115

NOTE.—No record for February; station abandoned July 31.

SCHOHARIE CREEK AT ERIE CANAL AQUEDUCT, NEW YORK.

The Erie Canal crosses Schoharie Creek between the Fort Hunter dam and Mohawk River. A gaging station was established at the aqueduct on May 2, 1900, by Prof. E. D. Walker. A current-meter measurement of the flow through the archways of the canal aqueduct was made by him on that day, the result being a discharge of 1,257 second-feet, with a gage height of 2.26 feet. Owing to cross currents above the aqueduct, the conditions are not favorable for meter measurements, and the station was abandoned on October 13. The record is chiefly of interest in connection with slope measurements of Mohawk River, described elsewhere (p. 42).

Daily gage height, in feet, of Schoharie Creek at Erie Canal aqueduct, New York, for 1900.

Day.	May.	June.	July.	Aug.	Sept.	Oct.	Day.	May.	June	July.	Aug.	Sept.
1	.....	1.38	0.60	0.67	0.54	0.53	17	1.58	0.82	0.58	1.43	0.65
2	2.24	1.47	.62	.65	.53	.54	18	1.52	.71	.56	1.32	.65
3	2.20	1.76	.58	.64	.54	.51	19	1.47	.82	.60	1.92	.65
4	2.14	1.62	.60	.63	.53	.53	20	1.98	.70	.61	.97	.65
5	2.10	.94	.60	.62	.54	.58	21	2.84	.72	.63	.78	.65
6	2.00	.95	.57	.61	.54	.61	22	2.43	.70	.65	.71	.65
7	1.10	.87	.56	.52	.53	.62	23	2.14	.67	.62	.68	.64
8	2.00	.90	.50	.52	.54	.58	24	1.93	.60	.68	.64	.62
9	1.97	1.90	.43	.51	.64	.54	25	1.87	.60	.71	.63	.53
10	2.00	1.84	.52	.50	.64	.52	26	1.80	.60	.73	.54	.56
11	1.10	1.80	.60	.52	.65	.59	27	1.67	.60	.78	.53	.57
12	1.97	1.40	.63	.53	.65	.56	28	1.40	.62	.72	.54	.53
13	1.89	1.68	.64	.52	.65	.57	29	1.47	.61	.60	.53	.52
14	1.76	1.40	.60	.57	.64	.....	30	1.45	.60	.63	.53	.53
15	1.70	1.13	.54	.54	.65	.....	31	1.40	.....	.64	.54	.....
16	1.67	1.20	.53	.53	.64	.....						

SCHOHARIE CREEK AT MILL POINT, NEW YORK.

The current-meter station was established at the Mill Point highway bridge on July 5, 1900, by Prof. E. D. Walker. The stream bed is stony and fairly permanent. The channel is of nearly constant width at all stages of the stream. The following current-meter measurements were made by Professor Walker:

July 5: Gage height, 0.64 foot; discharge, 87 second-feet.
August 22: Gage height, 0.70 foot; discharge, 141 second-feet.

Daily gage height, in feet, of Schoharie Creek at Mill Point, New York, for 1900.

Day.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Day.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	.....	0.65	0.35	0.50	0.55	2.10	17	0.60	0.95	0.35	0.40	0.65	1.65
2	.....	.65	.35	.50	.45	1.90	18	.65	1.45	.30	.45	.70	1.70
3	.....	.45	.35	.45	.60	1.80	19	.65	.95	.35	.45	.75	1.82
4	.....	.40	.30	.40	.60	1.70	20	.65	.72	.25	.45	.98	1.95
5	.....	.50	.25	.40	.50	3.02	21	.60	.70	.00	.40	1.45	2.30
6	0.60	.35	.22	.45	.40	2.60	22	.95	.75	.00	.30	1.42	2.10
7	.35	.40	.40	.35	.60	2.35	23	.90	.70	.00	.60	1.50	1.70
8	.70	.35	.30	.30	.60	2.25	24	.80	.60	.30	.60	1.35	1.88
9	.50	.30	.30	.15	.65	2.10	25	.85	.55	.35	.50	1.30	2.95
10	.80	.30	.25	.00	.85	1.80	26	.80	.50	.30	.70	1.72	3.70
11	.70	.40	.45	.00	.60	1.75	27	.80	.45	.30	.75	3.60	1.70
12	.70	.40	.45	.35	.60	1.75	28	.80	.50	.35	.52	2.75	2.05
13	.60	.45	.35	.45	.80	1.72	29	.70	.50	.45	.45	2.25	1.95
14	.60	.30	.35	.15	.90	1.70	30	.70	.55	.45	.52	2.15	1.50
15	.60	.25	.30	.30	.72	1.65	31	.65	.43	.....	.50	.....	1.60
16	.65	.40	.35	.30	.70	1.60							

SCHOHARIE CREEK AT SCHOHARIE FALLS, NEW YORK.

A dam and power plant have been erected by the Empire State Power Company, of Amsterdam, New York, at Schoharie Falls, 7 miles from Amsterdam. The dam is of masonry, backed with timber. It has a flat crest, nearly level, 380 feet long, 1 foot wide, and a slope on the upstream face of approximately 2½ to 1. The dam was completed early in the summer of 1900, and a record of the depth of water flowing over the main spillway was kept by the engineers of the company from July 18 to December 31. Some further data are needed,

however, for the final reduction of this record, which is reserved for future publication. The dam is about 1 mile above the current-meter station on Schoharie Creek, at the Mill Point highway bridge, and it is the intention to use the results of the meter measurements made there in the preparation of a calibration curve for the spillway of the dam.

Soon after the completion of the dam a weir of standard form was placed in an opening in the water-power canal embankment, at a point where the entire flow of the stream could be concentrated so as to pass over the gaging weir. The weir has a sharp crest, 25 feet in length, with two complete contractions, and the following observations of flow were computed from the observed depths by the Francis formula:

Flow over weir at Schoharie Falls dam.

Date.	Time of measurement.	Discharge.	Date.	Time of measurement.	Discharge.
1900.		*Second-feet.*	1900.		*Second-feet.*
June 25	11 a. m.	86.2	June 29	5 p. m.	86.2
June 26	11 a. m.	91.6	June 30	11 a. m.	92.9
Do	5 p. m.	91.5	Do	5 p. m.	92.9
June 27	11 a. m.	92.9	July 1	11 a. m.	92.9
Do	5 p. m.	92.9	Do	5 p. m.	91.5
June 28	11 a. m.	86.2	July 2	11 a. m.	91.6
Do	5 p. m.	91.5	Do	5 p. m.	92.9
June 29	11 a. m.	86.2	July 3	9 a. m.	93.4

The power plant at the dam contains two double horizontal Samson turbines, each 40 inches in diameter. The turbines are designed to work under a head of 42 feet, and are rated at 1,800 horsepower per pair. A wheel similar to these has been tested at Holyoke, and with the rating curve established, which shows the proportional discharge of the wheels for various depths of gate opening, the turbines can be used as water meters to determine the flow in the power canal. The plant was set in operation about January 1, 1901, and a gaging record is being kept, which shows the depth of flow over the crest of the dam and spillway, the discharge through the flood gates, and the amount of water used by the turbines. It should be stated that this stream is subject to extreme variations of flow, and that the weir measurements given above represent unusually low water. Plans have been made for a system of storage reservoirs to conserve the entire discharge of the stream and maintain a nearly uniform regimen of flow throughout the year.

MOHAWK RIVER AT SCHENECTADY, NEW YORK.

A current-meter station at Freeman's tollbridge, near Schenectady, was established by Prof. E. D. Walker February 1, 1899, and

remained under his supervision until October 1, 1900. A description of this station is contained in Water-Supply Paper No. 35, page 55. The following current-meter measurements were made by Professor Walker:

April 3, 1899: Gage height, 7.18 feet; discharge, 5,294 second-feet.
May 26, 1899: Gage height, 6.22 feet; discharge, 2,092 second-feet.
June 30, 1899: Gage height, 5.38 feet; discharge, 482 second-feet.
May 12, 1900: Gage height, 6.50 feet; discharge, 4,135 second-feet.
July 17, 1900: Gage height, 5.26 feet; discharge, 667 second-feet.
August 21, 1900: Gage height, 5.40 feet; discharge, 976 second-feet.

A rating curve for the cross section at this station has not yet been prepared.

Daily gage height, in feet, of Mohawk River at Schenectady, New York, for 1900.

Day.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	.....	7.40	5.50	5.22	5.38	5.35	5.20	5.55	7.80
2	.....	7.40	5.50	5.20	5.35	5.25	5.25	5.60	7.35
3	.....	7.10	5.50	5.22	5.25	5.22	5.20	5.58	7.00
4	.....	6.90	5.75	5.18	5.23	5.20	5.25	5.50	6.75
5	.....	6.80	5.65	5.15	5.25	5.10	5.20	5.45	8.75
6	.....	6.65	5.55	5.20	5.20	5.15	5.20	5.42	8.85
7	.....	6.45	5.60	5.20	5.15	5.23	5.02	5.35	8.15
8	11.15	6.20	5.40	5.20	5.18	5.15	5.10	5.35	8.00
9	10.05	6.45	5.60	5.25	5.10	5.18	5.18	6.40	7.70
10	8.85	6.70	6.45	5.35	5.08	5.05	5.10	7.05	7.25
11	8.18	6.70	6.00	5.40	5.10	5.10	5.10	6.60	6.75
12	7.90	6.50	5.75	5.35	5.10	5.15	5.18	6.15	6.15
13	8.25	6.25	5.60	5.32	5.10	5.10	5.15	5.85	6.70
14	8.15	6.10	5.55	5.30	5.15	5.10	5.25	5.80	6.85
15	7.80	6.05	5.50	5.30	5.25	5.08	5.15	5.75	6.20
16	8.10	6.00	5.50	5.28	6.25	5.10	5.18	5.70	6.00
17	8.50	6.05	5.45	5.29	5.90	5.05	5.20	5.65	5.85
18	10.65	6.00	5.42	5.20	5.80	5.08	5.20	5.50	5.95
19	12.05	6.95	5.35	5.23	5.62	5.00	5.42	5.60	5.90
20	11.90	6.05	5.22	5.35	5.45	5.05	5.35	5.80	6.00
21	11.68	6.50	5.25	5.35	5.40	5.08	5.20	6.95	6.15
22	10.85	6.20	5.25	5.40	5.32	5.10	5.20	7.60	6.20
23	11.65	6.10	5.35	5.70	5.25	5.10	5.18	7.30	6.10
24	11.90	5.95	5.22	5.58	5.20	5.70	5.20	7.00	6.10
25	10.05	5.90	5.20	5.43	5.25	5.53	5.28	6.70	6.70
26	9.35	5.75	5.20	5.38	5.28	5.45	6.15	8.48	8.00
27	8.60	5.70	5.18	6.15	5.22	5.35	6.00	11.65	7.60
28	8.25	5.55	5.20	6.05	5.20	5.22	5.70	11.35	7.05
29	7.72	5.50	5.22	5.72	5.25	5.28	5.65	10.05	6.70
30	7.45	5.45	5.20	5.55	5.20	5.22	5.75	8.50	6.40
31	.....	5.50	.....	5.50	5.45	.....	5.65	.....	6.25

MOHAWK RIVER AT REXFORD FLATS, NEW YORK.

This station, which is located at the New York State feeder dam 4 miles below Schenectady, is described in Water-Supply Paper No. 35, page 57.

The accompanying tables of daily and monthly mean flow include the amount diverted to the Erie Canal. They therefore represent the total inflow of Mohawk River at this point, which is considerably greater than the amount which passes downstream from the dam during the season of canal navigation. Prior to 1900 the amount of diversion to the Erie Canal was assumed to be 128 second-feet. During the present year a different method of estimating diversion to

the canal has been used. Current-meter measurements in the Rexford Flats feeder gave the following results:

Current-meter measurements in Rexford Flats feeder.

Date.	Hydrographer.	Flow in canal feeder.
		Second-feet.
October 27, 1898	W. D. Lockwood	128
June 25, 1900	R. E. Horton	272

These results were compared with the mean rate of evaporation from a water surface, as determined for several years at Rochester,[1] and it was found that an apparently constant relation existed between the two. The diversion for the remaining months of the canal season has been estimated from the observed evaporation, as follows:

Estimated monthly flow in Rexford Flats feeder.

	Second-feet.
May	200
June	260
July	290
August	270
September	220
October	148

The flow over the dam—that is, the amount of water passing downstream from Rexford Flats—compares with the total flow as follows:

Table showing relation of flow over Rexford Flats dam to total inflow of feeder.

Month.	Total inflow.	Flow below dam (inflow less diversion).
1900.	*Second-feet.*	*Second-feet.*
May	2,857	2,657
June	1,503	1,243
July	1,447	1,157
August	1,746	1,476
September	981	761

The water did not fall below the crest of the dam during the summer of 1900, so that it was not possible to make measurements of the leakage, or a new profile of the crest, which is greatly needed. During high water for several days of the year 1900 the water on the downstream side of the dam rose above the crest level and the dam was completely submerged. Experiments on the flow over a similar submerged weir are not available, and the high-water flows have been taken from the usual discharge curve. On the morning of February

[1] See annual reports of the executive board of the city of Rochester, 1892 to 1899, inclusive.

14, 1900, the water attained a depth of 9.25 feet on the crest of the dam, corresponding to a discharge of 55,700 second-feet, or 16.5 second-feet per square mile. This is the highest water observed since the record was started.

During 1900 the following discharge measurements were made by Prof. E. D. Walker, at Freeman's bridge, 3 miles above the Rexford Flats gaging station:

May 12: Gage height, 6.50 feet; discharge, 4,135 second-feet.
July 17: Gage height, 5.26 feet; discharge, 667 second-feet.
August 21: Gage height, 5.40 feet; discharge, 976 second-feet.

Daily discharge, in second-feet, of Mohawk River at Rexford Flats, New York, for 1898.

[Drainage area, 3,385 square miles.]

Day.	Dec.	Day.	Dec.	Day.	Dec.	Day.	Dec.
8	5,050	15	4,959	22	2,850	29	3,350
9	5,050	16	3,250	23	5,150	30	2,400
10	5,050	17	2,450	24	11,550	31	1,800
11	5,450	18	2,300	25	9,350		
12	4,200	19	2,675	26	7,350	Mean	4,471
13	3,625	20	2,550	27	4,450		
14	5,250	21	2,300	28	4,900		

Daily discharge, in second-feet, of Mohawk River at Rexford Flats, New York, for 1899.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	9,000	2,950	6,300	5,850	9,828	2,288	928	328	228	2,728	1,368	1,328
2	4,850	3,460	5,750	6,350	8,828	2,288	928	328	228	2,928	1,368	1,328
3	3,350	4,350	5,170	5,350	8,428	2,078	679	438	228	2,928	2,928	3,208
4	3,600	3,950	4,650	5,270	8,528	2,078	608	438	228	2,628	5,028	2,448
5	9,350	3,950	6,920	7,880	5,828	2,078	608	528	228	2,628	6,528	2,628
6	16,850	4,550	25,700	10,510	5,178	1,538	608	608	228	2,628	6,528	2,908
7	12,550	3,350	15,110	14,050	2,628	1,538	438	328	228	2,478	8,028	3,578
8	9,350	2,700	9,770	18,330	2,328	1,538	438	278	228	2,328	9,028	4,548
9	7,925	2,600	8,450	21,770	2,328	1,378	608	278	228	2,178	7,928	6,078
10	4,850	2,400	8,350	13,340	2,178	1,378	528	278	228	1,428	4,828	8,328
11	4,250	2,100	7,000	15,590	2,178	1,128	438	278	228	728	2,778	8,968
12	3,600	2,850	9,160	18,600	2,428	1,128	438	278	228	728	2,178	10,028
13	2,850	3,350	20,670	24,040	2,328	928	378	278	228	728	1,578	10,828
14	3,825	4,750	14,000	29,550	2,078	928	328	278	228	728	1,928	15,828
15	4,650	4,650	10,950	30,630	1,828	1,088	328	278	238	1,228	2,078	21,358
16	5,400	3,850	10,050	29,350	2,428	1,088	328	278	278	1,578	1,578	24,228
17	8,750	3,460	8,350	22,000	1,978	2,928	758	278	278	1,828	1,368	19,128
18	9,200	2,700	5,750	20,550	1,778	3,528	928	278	278	1,928	1,368	13,128
19	5,400	2,400	5,470	20,350	1,678	4,628	678	278	278	1,428	1,128	8,128
20	4,475	2,400	8,250	21,150	4,778	5,128	678	228	278	1,348	1,128	10,228
21	2,850	2,250	13,560	21,628	6,028	2,628	438	228	278	1,348	1,228	6,328
22	2,850	2,600	12,600	19,228	6,828	3,228	328	228	278	1,348	1,478	4,928
23	2,750	5,450	11,400	17,878	6,728	2,168	328	228	278	1,228	1,578	4,578
24	3,050	10,150	6,460	17,878	5,428	2,168	308	228	278	1,148	1,578	4,578
25	3,450	5,750	7,050	17,878	4,928	2,078	288	228	278	1,128	1,468	2,428
26	4,300	5,550	6,740	17,028	4,378	1,538	288	278	3,228	1,028	1,368	2,928
27	4,475	4,650	5,370	16,428	3,128	1,538	328	278	9,178	828	1,138	2,928
28	4,950	7,000	4,750	16,428	2,228	1,538	438	238	6,328	728	1,128	2,928
29	4,750	-------	4,650	15,668	1,828	1,428	328	208	1,678	1,228	1,128	2,628
30	5,575	-------	5,560	11,148	2,778	1,538	378	208	2,778	1,348	1,368	2,428
31	4,850	-------	5,170	-------	2,778	-------	338	208	-------	1,348	-------	2,228
Mean	5,739	3,935	9,004	17,057	4,084	2,014	498	294	980	1,608	2,824	7,001

Daily discharge, in second-feet, of Mohawk River at Rexford Flats, New York, for 1900.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.
1	2,080	2,850	4,650	8,750	5,500	1,470	1,500	1,970	1,920
2	1,500	1,860	4,650	9,400	4,800	1,470	940	1,830	1,670
3	1,500	2,030	4,450	11,150	4,150	1,580	940	1,720	1,670
4	1,380	3,830	4,450	11,400	3,300	1,710	940	1,720	1,540
5	1,380	4,450	4,450	12,850	2,850	1,710	820	1,720	1,430
6	1,500	4,950	4,800	13,550	3,000	1,470	750	1,970	1,430
7	1,860	5,550	3,325	13,850	3,450	1,470	750	1,720	1,430
8	2,250	7,150	3,000	14,650	3,300	1,710	820	1,970	1,020
9	2,550	7,250	2,250	15,070	3,800	1,710	1,090	1,970	1,020
10	2,550	7,950	2,250	15,950	4,400	1,960	1,090	1,970	1,120
11	1,860	10,730	1,860	17,150	3,900	2,240	1,290	1,970	1,220
12	1,610	13,050	1,610	18,300	3,650	2,910	1,500	1,970	1,020
13	1,500	27,250	1,500	15,650	3,300	2,240	1,500	1,970	870
14	1,500	45,050	1,500	13,850	3,300	1,580	1,290	2,250	870
15	1,860	24,850	1,370	9,390	3,800	1,580	1,290	2,390	430
16	2,700	14,850	1,280	9,000	3,300	1,580	1,290	2,470	430
17	3,830	9,650	1,160	12,350	3,000	1,710	1,090	5,020	680
18	6,500	5,750	1,160	20,750	2,850	1,710	1,090	1,830	680
19	10,730	3,150	1,750	27,450	2,700	1,470	1,090	1,830	750
20	21,850	2,470	3,150	25,550	2,250	1,260	1,090	1,970	870
21	45,750	3,830	3,950	26,250	2,250	1,260	1,400	1,070	870
22	27,850	5,150	4,800	28,370	1,750	1,260	1,610	1,070	680
23	28,450	6,690	4,800	26,050	1,650	1,060	1,740	1,070	1,020
24	21,750	8,750	5,150	21,730	1,650	1,060	1,990	1,270	1,020
25	13,300	7,850	5,950	17,050	1,650	1,060	2,270	1,170	750
26	9,250	6,150	6,900	11,900	1,500	1,060	2,270	1,070	680
27	6,100	5,150	7,700	11,400	1,900	1,060	2,490	920	680
28	5,150	4,650	7,700	9,850	1,650	910	2,380	1,000	540
29	5,150	.....	8,430	8,750	1,425	910	2,270	1,000	600
30	4,450	.....	8,550	7,250	1,425	910	2,270	1,070	540
31	4,250	.....	8,750	.....	1,425	.....	1,990	1,170	.....
Mean	7,860	9,032	4,235	14,996	2,857	1,503	1,447	1,746	981

NOTE.—Records for October, November, and December are not available at present.

MOHAWK RIVER NEAR DUNSBACH FERRY, NEW YORK.

A gaging record has been kept at the dam of the West Troy Water Company near Dunsbach Ferry, 9 miles from the mouth of Mohawk River. The dam is of masonry, and has a flat granite crest 5.5 feet wide, standing 0.75 foot higher at the crest lip than at the upstream edge. An island in the center of the stream divides the dam into two sections. The crest of the right wing, at the upper end of the island, is 380 feet long, with an average elevation of 174.15 feet. The crest of the left wing, at the lower end of the island, is 280 feet in length, and stands at an elevation of from 173.46 to 173.50 feet. Openings which existed in the dam during a part of the time the record has been kept should be taken into consideration in computing the flow.

The crest of the dam at the present time stands nearly level. The elevation of the zero of the crest gage is 172 feet. Readings are taken twice daily, usually between 6 and 7 a. m. and between 5 and 6 p. m. The mean of the readings taken for each day is given in the accompanying table.

In the adjacent pumping station are two Eclipse turbines, built by Stout, Mills & Temple, of Dayton, Ohio. One is 68 and the other 72 inches in diameter. They are run with gates wide open, under a head of from 7 to 8 feet.

The record at Dunsbach Ferry was established primarily for the purpose of checking a system of levels which were run for the United States Board of Engineers on Deep Waterways, by D. J. Howell, C. E., who furnished the record to the Geological Survey. No record was kept from April 1, 1899, to July 31, 1900.

The drainage area above the station at Dunsbach Ferry is 3,422 square miles.

Daily gage height, in feet, of Mohawk River near Dunsbach Ferry, New York, for 1898.

Day.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	.....	4.00	3.20	3.00	1.40	2.50	2.80	2.40	3.60	3.20
2	.....	3.60	3.10	2.70	1.00	5.50	2.40	2.40	3.40	3.10
3	.....	3.20	3.20	2.40	.60	2.20	2.20	2.40	3.20	3.00
4	.....	3.00	3.60	2.40	.40	2.70	2.20	2.40	3.20	3.00
5	.....	2.80	4.20	2.40	.20	2.60	2.10	2.60	3.00	3.10
6	.....	2.60	4.20	2.20	.00	3.50	2.20	5.80	3.00	3.40
7	.....	2.50	4.20	2.00	—.10	3.00	2.40	4.20	3.00	3.30
8	.....	2.60	3.50	1.80	—.10	2.40	2.60	3.30	3.00	3.20
9	.....	2.60	3.10	1.70	—.10	2.00	3.00	3.20	3.10	3.20
10	.....	2.40	2.60	1.70	—.20	1.00	2.70	3.10	3.30	3.20
11	.....	2.40	2.60	1.80	—.30	1.80	2.30	2.80	8.40	3.10
12	7.60	2.40	2.40	1.80	—.40	1.50	2.00	2.70	6.80	3.00
13	8.20	2.40	3.60	2.70	—.60	1.20	1.80	2.70	5.50	2.90
14	8.40	2.50	3.80	3.40	—.60	1.00	1.60	2.80	4.90	2.80
15	7.50	2.60	3.40	3.10	—.00	.90	1.40	3.60	4.40	2.80
16	6.40	2.80	3.00	2.90	—.60	.80	1.30	4.90	4.00	2.80
17	5.60	3.00	2.80	2.60	—.80	1.40	1.30	4.30	3.60	2.80
18	5.60	2.80	2.00	2.20	—.70	2.40	1.20	3.80	3.60	2.80
19	5.30	2.70	2.40	2.00	—.70	2.50	1.20	3.40	3.60	2.80
20	6.50	2.80	2.50	2.00	1.70	2.90	1.80	3.30	4.80	2.80
21	6.20	3.20	2.00	2.60	2.20	3.40	2.20	3.30	4.30	2.80
22	5.40	3.30	2.60	2.20	1.90	2.90	2.00	3.40	3.80	3.00
23	4.80	3.40	2.40	2.20	1.70	2.60	2.00	4.00	3.70	3.60
24	4.80	4.70	2.50	2.00	.80	3.00	2.20	4.20	3.80	4.60
25	4.10	7.00	3.30	1.80	—.20	6.10	3.00	4.00	3.80	4.50
26	3.80	6.80	4.80	1.70	.00	6.40	3.20	3.80	3.50	4.20
27	3.60	6.20	5.20	1.70	1.20	5.20	2.80	4.40	3.50	3.80
28	3.60	5.00	5.20	1.60	1.80	4.00	2.80	5.40	3.00	3.40
29	4.00	4.30	4.00	1.40	1.60	3.20	2.60	5.00	3.00	3.00
30	4.80	3.70	3.40	1.40	1.20	2.80	2.50	4.30	3.00	2.80
31	4.70	.....	3.20	.....	.60	2.80	.....	3.80	.....	3.20

Daily gage height, in feet, of Mohawk River near Dunsbach Ferry, New York, for 1899.

Day.	Jan.	Feb.	Mar.	Day.	Jan.	Feb.	Mar.
1	3.90	2.60	4.00	17	4.20	2.20	4.40
2	3.40	2.60	3.80	18	4.00	2.30	3.80
3	3.20	2.60	3.60	19	3.80	2.40	3.60
4	3.30	2.60	3.30	20	3.40	2.40	4.20
5	4.00	2.60	3.40	21	3.20	2.60	4.40
6	5.80	2.60	6.60	22	3.00	3.00	4.00
7	5.00	2.50	6.00	23	3.00	3.40	3.80
8	4.40	2.40	4.80	24	3.00	4.60	4.00
9	4.00	2.20	4.20	25	3.10	3.90	4.40
10	3.60	2.20	4.10	26	3.20	3.60	4.00
11	3.20	2.20	3.80	27	3.20	3.50	3.80
12	3.20	2.20	3.90	28	3.00	3.80	3.60
13	3.00	2.20	6.00	29	2.80	.....	3.60
14	3.10	2.20	5.60	30	2.70	.....	3.60
15	3.60	2.20	4.90	31	2.70	.....	3.50
16	4.40	2.20	4.50				

Daily gage height, in feet, of Mohawk River near Dunsbach Ferry, New York, for 1900.

Day.	Aug.	Sept.	Oct.	Nov.	Dec.	Day.	Aug.	Sept.	Oct.	Nov.	Dec.
1	2.30	2.20	2.00	2.40	3.75	17	2.70	1.80	2.00	2.45	2.85
2	2.20	2.20	2.00	2.40	3.70	18	2.60	1.80	2.20	2.35	2.80
3	2.10	2.10	2.00	2.45	3.55	19	2.40	1.60	2.20	2.40	2.90
4	2.00	2.00	2.00	2.35	3.50	20	2.30	1.60	2.20	2.50	2.90
5	2.10	2.00	2.00	2.30	4.80	21	2.20	1.70	2.00	3.50	2.85
6	2.00	1.80	2.00	2.20	4.80	22	2.20	1.80	2.00	4.00	2.80
7	2.00	1.80	2.00	2.15	4.40	23	2.00	2.00	2.00	3.80	2.90
8	1.90	1.80	1.90	2.20	4.40	24	2.00	2.50	2.00	3.45	2.90
9	1.80	1.90	1.80	2.85	4.20	25	2.00	2.40	2.00	3.30	3.35
10	1.80	2.00	1.70	3.55	3.70	26	2.20	2.20	2.75	4.10	4.25
11	1.80	2.00	1.70	3.10	3.25	27	2.20	2.20	2.75	6.50	3.75
12	1.80	2.20	1.80	2.90	3.00	28	2.10	2.20	2.50	6.35	3.55
13	1.80	1.70	1.80	2.75	3.25	29	2.00	2.00	2.45	5.50	3.35
14	1.90	1.70	1.80	2.55	3.20	30	2.00	2.00	2.50	4.50	3.20
15	2.20	1.70	1.90	2.50	3.10	31	2.20	.....	2.40	.....	3.10
16	2.80	1.70	1.90	2.50	2.95						

INDIAN RIVER AT INDIAN LAKE DAM, NEW YORK.

Indian River, a tributary of the upper Hudson, has a precipitous forested mountain area of 146 square miles in eastern Hamilton County. In 1898 a masonry storage dam was built at the foot of Indian Lake, replacing the lumbermen's dam which was formerly there, and raising the level of the artificial lake so formed 23 feet. The storage capacity of the present lake is 5,000,000,000 cubic feet, the area of its water surface is 5,035 acres, and the elevation of the spillway crest above mean tide is 1,650 feet. The dam was built by a federation of water-power users on Hudson River, in cooperation with the State of New York, under the direction of Mr. George W. Rafter, engineer in charge, the primary object being to store flood waters from this drainage area, to be turned into the Hudson during the low-water period of the year, thereby equalizing to some extent the flow of that stream. Water is also used for sluicing logs during the river driving season.[1]

Since July 22, 1900, a gaging record has been kept at the dam, with a view to determining the total outflow from this reservoir. The facts recorded are the elevation of the water surface in the reservoir, the depth of water flowing over the spillway or flashboards, the depth of opening and the head on the main and subsidiary logways, and the depth of the opening and the effective head on each of the 5-foot sluice gates. These data enable a calculation of the outflow from the reservoir to be made, and, by comparison with gaging records kept on Hudson River at Fort Edward and Mechanicsville, the effect of storage on the low-water flow of the latter river can be determined.

A meteorological station has been established at the dam by the United States Weather Bureau, and records are kept of the rainfall, temperature, etc. The regimen of flow of Indian River below the dam is largely artificial, though in the course of a year or more the total annual run-off of the drainage area will appear in the stream,

[1] Engineering News, May 18, 1899.

and it is hoped, in the course of time, to determine the relation between the rainfall and run-off of what constitutes a rather typical Adirondack watershed.

When the reservoir is full the excess of inflow passes over the spillway, which has a level crest 106.05 feet long in the clear. To facilitate the calculation of discharge over this spillway a series of experiments was made at Cornell University June 6, 1899, on a full-sized model of the spillway section (6.58 feet long), from which the proper coefficients of discharge have been determined.[1]

The discharge through the two 5-foot sluice gates, provided for the purpose of drawing the water down as required, is calculated from the observed head and from the area of the lune-shaped gate orifices by the ordinary formula, the value of the coefficients of discharge to be applied being checked by current-meter measurements made at a convenient bridge below the dam.

A meter measurement of the flow at this point on October 19, 1900, showed the rate of draft from the reservoir to be 451 second-feet, both sluice gates being wide open and under an effective head of 11 feet. A measurement of Hudson River at Mechanicsville, made on the afternoon of the following day, October 20, showed the total flow at that point to be 1,871 second-feet.

The following table shows the stage of the Indian Lake reservoir during the year 1900, the depth being measured from the base of the invert of the 5-foot discharge tunnels, also the estimated storage capacity of the reservoir at different depths:

Estimated storage capacity of Indian Lake reservoir.

Water surface.	Elevation.	Area.	Storage capacity.
	Feet.	*Acres.*	*Cubic feet.*
Original lake	1,616–17	1,000	.....
Lumbermen's dam	1,627	3,007	800,000,000
Crest of present dam	1,650	5,035	4,468,000,000
Top of flashboards of present dam	1,651.1	.....	5,000,000,000

Depth of water, in feet, in Indian Lake reservoir, New York, for 1900.

Day.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Day.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	.....	29.42	25.01	16.00	8.62	.....	17	.....	27.00	20.50	11.83	6.33	.....
2	.....	29.17	25.00	15.75	8.37	9.50	18	.....	27.00	20.08	11.54	6.17	10.83
3	.....	28.92	24.83	15.46	8.17	.....	19	.....	27.00	19.75	11.25	6.08	.....
4	.....	28.75	24.58	15.17	8.00	.....	20	.....	27.00	19.42	11.82	6.33	.....
5	.....	28.58	24.33	14.87	7.79	.....	21	.....	27.00	19.17	10.79	6.50	.....
6	.....	28.33	24.17	14.58	7.58	.....	22	33.50	27.00	18.87	10.54	6.58	11.08
7	.....	28.25	23.92	14.42	7.42	.....	23	33.00	26.92	18.54	10.33	6.58	.....
8	.....	28.08	23.58	14.08	7.33	.....	24	32.50	26.75	18.25	10.12	.....	11.25
9	.....	27.92	23.29	13.79	7.33	10.25	25	32.17	26.63	17.92	9.96	.....	.....
10	.....	27.83	22.00	13.50	7.29	.....	26	31.92	26.37	17.58	9.79	.....	11.58
11	.....	27.67	22.71	13.21	7.25	.....	27	31.25	26.17	17.25	9.58	.....	.....
12	.....	27.33	22.37	12.92	7.17	10.42	28	30.83	26.00	16.92	9.38	.....	.....
13	.....	27.29	21.00	12.67	7.00	.....	29	30.42	25.83	16.67	9.17	.....	11.83
14	.....	27.25	21.63	12.42	6.87	.....	30	30.08	25.58	16.33	8.96	9.25	.....
15	.....	27.21	21.25	12.25	6.71	.....	31	29.75	25.33	.....	8.83	.....	12.08
16	.....	27.12	20.88	12.08	6.50	10.75							

[1] Proc. Am. Soc. Civ. Engs., March, 1900, p. 288.

SCHROON RIVER AT WARRENSBURG, NEW YORK.

A gaging record was established at the dam of the Schroon River Pulp Company, 2 miles below Warrensburg, by George W. Rafter, member American Society of Civil Engineers, November 1, 1895, in connection with the upper Hudson storage surveys.[1] The conditions at the Warrensburg gaging station are somewhat peculiar. During ordinary water an attempt is made to turn the entire flow of the stream, less leakage, through the water wheels, which run 24 hours a day, Sundays excepted. This is accomplished by the use of flashboards and by draft from the storage impounded by the Starbuckville dam. During extreme low water the mill is shut down altogether. As a rule, no water passes over the dam at this time, the entire flow leaking through. A balance is maintained between the inflow and the outflow by fluctuations in the pond level, thereby varying the pond storage and also the head on the leaks. As no record is kept when the mill is not running, it has been necessary to estimate the low-water flow, which was taken at 150 second-feet in 1899, this being the assumed leakage of the Starbuckville dam.[2] The apparently uniform regimen of the stream during considerable periods may be accounted for as the result of draft and storage from the Starbuckville dam.

A current-meter measurement of the leakage of the dam, flume, and flashboards at the Schroon River Pulp Company's mill was made on August 9, 1900, in the open channel about a half mile below the dam. The flow at that point was found to be 285 second-feet, which amount has been taken as the low-water flow and leakage during the present year (1900). The dam is of timber, and was considered nearly watertight when built, but there is evidence that the leakage has increased year by year.

The flow over the dam without flashboards has been taken from a diagram deduced from experiments made at Cornell University on a weir having a similar cross section. The flow over the flashboards has been calculated by means of Francis's formula, with a constant coefficient of 3.33.

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[1] Ann. Rept. State Engineer and Surveyor of New York, 1895, p. 118; also Water-Supply and Irrigation Paper U. S. Geol. Survey No. 35, p. 58.

[2] Report on a water supply from the Adirondack Mountains for the city of New York, by Geo. W. Rafter: Rept. of Merchants' Association of New York on the water supply of the city of New York, p. 337.

Daily discharge, in second-feet, of Schroon River at Warrensburg, New York, for 1899.

[Drainage area, 563 square miles.]

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	*641	478	478	708	4,854	1,506	304	150	150	*381	1,468	708
2	641	478	478	*708	4,854	1,506	*304	150	150	381	1,468	708
3	641	478	478	708	4,854	1,506	304	150	*150	381	1,468	*708
4	641	478	478	708	4,854	*1,367	304	150	150	381	1,468	708
5	641	*478	*478	708	4,854	1,228	304	150	150	478	*1,468	708
6	641	478	478	708	4,854	1,228	304	*150	150	478	1,468	708
7	608	478	478	708	*4,487	1,228	304	150	150	478	1,468	708
8	*575	478	478	708	4,121	1,228	304	150	150	*478	1,468	708
9	575	478	478	*1,999	4,121	1,228	*304	150	150	478	1,318	708
10	575	478	478	3,291	4,121	1,228	304	150	*150	478	1,318	*708
11	575	478	478	3,291	4,121	*1,228	304	150	150	478	1,171	708
12	575	*478	*478	3,291	4,121	1,228	304	150	150	478	*1,106	708
13	575	478	478	3,291	3,493	1,228	150	*150	150	478	1,041	931
14	608	478	478	3,291	*3,307	1,228	150	150	150	478	983	983
15	*641	478	478	3,291	3,121	1,228	150	150	150	*478	901	1,041
16	641	478	478	*3,362	3,121	1,228	*150	150	150	478	861	1,101
17	641	478	478	3,443	2,931	1,228	150	150	*150	478	841	*1,127
18	641	478	559	3,443	2,931	*1,228	150	150	150	478	841	1,153
19	641	*478	*641	3,443	2,931	1,228	150	150	150	478	*841	1,153
20	641	478	641	3,443	2,381	1,228	150	*150	381	478	841	1,153
21	608	478	641	3,443	*2,181	1,228	150	150	381	478	841	1,153
22	*575	478	641	3,443	1,981	1,228	150	150	381	*478	841	1,153
23	575	478	641	*4,273	1,981	1,228	*150	150	381	478	841	1,153
24	575	478	718	5,103	1,981	970	150	150	*381	478	841	*1,153
25	575	478	718	5,103	1,981	*675	150	150	381	478	841	1,153
26	575	*478	*718	5,103	1,981	381	150	150	381	478	*774	1,153
27	575	478	718	5,103	1,506	381	150	*150	381	478	708	1,101
28	575	478	718	5,103	*1,448	381	150	150	381	478	708	1,101
29	*575	.....	718	5,103	1,391	381	150	150	381	*478	708	1,041
30	575	.....	718	*5,103	1,391	381	*150	150	381	478	708	1,041
31	575	.....	718	.....	1,391	.....	150	150	.....	478	.....	*1,041
Mean	606	478	564	2,877	3,150	1,093	210	150	234	462	1,047	943

* Sunday.

Daily discharge, in second-feet, of Schroon River at Warrensburg, New York, for 1900.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	810	810	1,360	*1,162	3,965	1,085	*885	285	285	285	517	989
2	810	810	1,290	1,215	2,885	1,217	810	285	*285	535	517	*285
3	810	810	1,230	1,315	2,625	*1,349	810	285	285	535	517	989
4	810	*810	*1,204	1,395	2,305	1,565	810	285	285	535	*285	989
5	810	810	1,178	1,535	2,155	1,990	810	*285	285	517	535	989
6	810	810	1,140	1,535	*2,045	1,890	810	285	285	517	535	989
7	*810	810	1,123	1,741	1,965	1,890	810	285	285	*285	535	989
8	810	810	1,123	*1,947	1,748	1,890	*810	285	285	535	517	989
9	810	810	1,123	2,155	1,715	1,890	616	285	*285	535	517	*285
10	810	810	1,123	2,155	1,715	*1,725	616	285	285	535	517	989
11	810	*1,895	*1,123	2,155	1,715	1,565	616	285	285	517	*285	989
12	810	2,035	1,123	2,225	1,715	1,425	616	*285	285	517	535	989
13	810	1,165	1,123	2,305	*1,670	1,300	616	806	285	517	535	989
14	*810	1,945	1,123	2,465	1,625	1,290	616	806	285	*285	535	989
15	810	1,945	1,123	*2,765	1,605	1,300	*616	806	285	535	517	989
16	810	1,945	1,123	3,065	1,585	1,300	539	806	*285	535	517	*285
17	810	1,945	1,123	3,365	1,545	*1,187	539	806	285	535	517	708
18	810	*1,892	*1,120	3,545	1,535	1,075	539	806	285	517	*285	708
19	810	1,840	1,117	4,115	1,525	963	365	*285	285	517	401	708
20	810	1,795	1,107	5,365	*1,470	963	365	806	285	517	535	708
21	*810	1,710	1,100	7,010	1,415	963	285	806	285	*285	535	708
22	810	1,608	1,089	*7,745	1,375	963	*285	806	285	535	535	708
23	810	1,608	1,085	6,945	1,355	963	285	806	*285	535	574	285
24	810	1,608	1,085	6,945	1,295	*963	285	612	285	535	285	*285
25	810	*1,571	*1,085	6,210	1,295	963	285	612	285	517	*285	708
26	810	1,535	1,085	6,685	1,125	963	285	*285	285	517	806	708
27	810	1,475	1,155	6,315	*1,025	963	285	285	285	517	806	708
28	*810	1,435	1,130	5,365	1,085	963	285	285	285	*285	806	708
29	810	.....	1,110	*5,095	1,085	963	*285	285	285	535	806	708
30	810	.....	1,110	4,825	1,085	963	285	285	*285	535	806	285
31	810	.....	1,110	.....	1,085	.....	285	285	.....	535	.....	*708
Mean	810	1,380	1,140	3,688	1,688	1,280	528	474	285	488	530	[illegible]

* Sunday.

HUDSON RIVER AT FORT EDWARD, NEW YORK.

This station, located at the dam of the International Paper Company, was established in 1895, by George W. Rafter, member American Society of Civil Engineers, in connection with upper Hudson storage surveys. The dam is of timber, on slate rock foundation, and has but little leakage. The crest is straight, very nearly level, and 587.6 feet in length. The zero of the crest gage stands at the level of the lip of the dam proper. Flashboards from 15 to 18 inches in height are usually maintained on the dam. A record is kept of the height of the flashboards and of the times of their setting and removal.

In the adjoining mill there are 62 water wheels, nearly all of modern types, which have been tested at the Holyoke flume. A record is kept of the daily run of each wheel, in hours, as well as of the working head, which usually is 19 feet. The discharge through the turbines is estimated from diagrams expressing the flow as a function of the working head and the number of wheel-hours run.

In the winter of 1896–97 a flood spillway was cut around the south end of the dam, over which the water flows whenever it reaches the level of the crest of the flashboards. The profile of the spillway is very irregular, and causes some uncertainty in the calculated flows during high water.

When the flashboards are off the flow is computed by means of the East Indian Engineers' formula;[1] when they are on the flow is computed by Francis's well-known formula for sharp-crested weir. During the dry season little water passes over the dam, the entire flow being employed to drive the turbines. A current-meter measurement was made at the highway bridge below the dam on July 26, 1900, and the flow was found to be 2,704 second-feet.

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[1] Mullin's Irrigation Manual.

Daily discharge, in second-feet, of Hudson River at Fort Edward, New York, for 1899.

[Drainage area, 2,800 square miles.]

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	*2,415	2,822	3,744	6,514	23,033	3,106	583	363	923	*2,181	9,268	1,941
2	2,619	2,822	3,744	*1,600	23,033	3,106	*945	363	923	2,181	11,948	1,941
3	2,619	2,502	3,744	4,378	23,033	3,106	945	363	*20	1,785	12,250	*1,280
4	2,619	2,606	6,154	4,378	19,689	*1,356	945	363	1,186	1,785	9,403	4,267
5	2,619	*945	*2,415	4,378	19,689	2,218	583	363	1,186	1,211	*14,930	4,307
6	2,877	1,839	5,444	4,378	21,949	2,218	289	*(a)	1,184	912	9,402	4,307
7	2,877	1,539	5,444	4,654	*12,340	2,218	743	(a)	920	1,172	5,668	4,243
8	*4,335	1,539	5,444	4,654	21,940	2,218	762	(a)	920	*20	7,974	4,243
9	4,320	1,539	5,444	*1,560	21,949	2,218	*100	(a)	920	1,008	7,951	4,243
10	3,896	1,539	5,444	4,334	8,283	2,138	1,547	(a)	*20	1,112	6,149	*1,016
11	3,872	1,162	5,444	9,202	6,549	*1,356	1,547	(a)	975	1,182	6,149	2,194
12	2,944	*942	*3,025	9,967	7,049	1,917	1,817	(a)	953	1,182	*1,860	2,369
13	3,225	2,109	5,444	9,967	2,049	1,917	1,817	*(a)	867	1,182	8,923	9,675
14	4,300	2,109	5,444	17,428	*6,900	1,912	1,817	574	631	919	3,817	11,410
15	*2,415	2,109	5,444	17,406	7,049	1,912	1,817	574	620	*20	3,135	11,410
16	5,368	2,109	5,315	*20,095	5,204	1,860	*455	574	577	1,177	3,133	8,130
17	5,388	2,109	4,335	21,222	5,204	1,720	1,817	916	*20	575	3,133	*5,882
18	5,388	1,504	4,335	21,222	5,204	*945	1,817	916	624	576	3,133	8,229
19	5,388	*1,150	*3,344	21,800	5,204	1,683	1,817	916	624	826	*20	8,229
20	5,388	1,682	6,099	27,059	8,617	1,454	1,817	*20	577	861	2,900	8,574
21	5,388	1,682	6,229	29,545	*700	988	1,645	1,176	954	1,184	2,900	8,574
22	*1,356	1,682	6,229	29,618	4,864	752	1,645	1,176	1,024	*20	2,900	8,574
23	3,491	2,012	6,229	*25,640	4,864	590	*465	1,176	1,024	1,446	2,580	8,229
24	3,776	2,368	5,703	32,158	4,184	590	1,368	916	*20	1,439	2,311	*2,415
25	3,618	1,600	5,703	32,159	4,282	*747	1,300	916	1,184	923	2,044	2,415
26	3,623	*1,600	*2,415	29,619	3,901	1,329	1,103	916	3,404	918	*20	4,839
27	3,623	2,853	5,444	29,619	3,901	894	1,103	*574	4,889	919	2,311	4,234
28	3,041	2,842	5,444	29,620	*1,356	888	1,275	1,176	4,889	919	2,206	4,234
29	*925	.....	5,506	27,032	3,104	585	1,275	576	4,181	*20	2,206	4,234
30	2,816	.....	5,506	*23,120	3,104	583	*20	576	4,181	1,183	2,206	4,234
31	2,816	.....	5,506	.....	3,104	.....	461	948	.....	1,183	.....	*20
Mean	3,527	1,902	5,005	16,811	9,561	1,617	1,150	714	1,347	1,033	5,008	5,157

* Sunday. a No record.

Daily discharge, in second-feet, of Hudson River at Fort Edward, New York, for 1900.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
*1	3,001	3,481	2,879	*3,237	12,486	4,924	*175	1,927	1,044	827	1,620	5,586
2	1,957	3,221	4,242	4,932	11,899	4,924	827	1,401	*956	827	1,266	*5,611
3	1,957	3,221	4,362	5,261	10,776	*3,422	827	1,719	190	827	1,406	7,697
4	1,957	*1,810	*3,019	5,701	11,626	4,921	30	1,332	1,445	827	*302	4,421
5	1,957	2,961	4,462	5,701	9,646	5,094	1,216	*1,087	827	827	1,462	4,421
6	2,053	2,447	3,215	7,376	*3,015	4,674	931	1,256	715	827	834	4,183
7	*1,401	2,447	3,859	10,566	7,351	4,419	821	1,087	827	*20	1,226	4,661
8	2,217	2,447	3,741	*11,360	8,246	4,440	*1,215	1,350	827	1,466	1,006	4,181
9	2,217	3,221	3,471	14,316	5,836	4,539	1,288	1,579	*827	1,216	1,119	*7,291
10	2,217	3,889	4,208	11,636	8,246	*1,367	1,255	1,740	1,211	1,216	2,203	2,454
11	2,217	*2,062	*3,259	11,636	6,608	3,859	827	1,729	1,211	1,258	*875	2,061
12	2,217	4,136	3,879	11,636	6,608	3,201	1,260	*1,053	1,211	1,238	2,203	2,418
13	2,217	7,369	3,659	10,776	*4,450	2,924	841	2,271	827	1,366	2,203	2,636
14	*1,571	16,615	3,723	10,776	5,361	2,921	1,467	2,565	1,579	*872	1,883	2,7[illegible]
15	2,217	17,747	3,759	*9,202	6,378	2,981	*803	3,019	1,293	1,818	1,947	2,13[illegible]
16	2,217	18,499	3,571	10,776	7,348	2,268	2,171	3,233	*686	1,504	1,691	*985
17	2,217	18,975	3,095	12,946	5,993	*795	1,248	3,347	1,233	1,306	2,947	2,2[illegible]
18	2,217	*13,045	*1,302	17,076	6,987	2,941	1,220	2,989	1,233	1,450	*1,133	1,858
19	2,217	15,531	3,479	23,626	5,961	1,677	934	*1,420	1,112	1,246	2,040	1,940
20	2,217	9,340	3,479	31,495	*3,015	1,957	1,360	2,242	1,112	1,224	1,901	1,65[illegible]
21	*1,401	9,141	4,830	34,899	7,342	2,407	1,268	1,250	1,211	*872	4,863	2,13[illegible]
22	5,858	7,611	4,942	*34,470	5,967	1,915	*30	1,534	1,175	1,611	7,074	2,13[illegible]
23	7,312	7,013	4,942	43,900	6,336	1,979	1,308	977	*912	1,040	10,213	*1,76[illegible]
24	7,577	6,259	4,942	36,061	5,111	*20	1,338	940	1,612	1,118	6,482	1,590
25	6,991	*5,609	*2,911	30,945	3,677	2,607	2,033	901	1,661	1,106	*3,260	3,170
26	6,991	4,694	4,742	26,635	4,912	2,191	1,467	*95	1,557	1,251	6,268	2,65[illegible]
27	5,862	4,232	4,742	23,536	*415	1,178	3,441	928	1,329	1,773	.....	3,13[illegible]
28	*2,041	4,045	4,466	18,480	4,366	2,181	3,263	928	1,435	*804	.....	2,4[illegible]
29	5,212	.....	4,882	*14,250	4,319	1,241	*175	1,487	1,180	2,703	.....	2,4[illegible]
30	4,287	.....	4,942	14,210	3,372	1,069	1,987	1,531	*865	2,447	.....	2,65[illegible]
31	3,571	.....	4,942	.....	3,460	.....	1,656	1,791	.....	1,647	.....	2,65[illegible]
Mean	3,211	7,074	3,934	16,914	6,358	2,834	1,248	1,652	1,110	1,243	2,670	3,19[illegible]

* Sunday.

HUDSON RIVER AT MECHANICSVILLE, NEW YORK.

A record of the flow of Hudson River at Mechanicsville has been kept by the Duncan Company since December, 1888. The record includes two daily readings of the depth of water on the crest of the dam and a continuous record of the water wheels in the adjoining paper mill. The accompanying tables give the daily mean flow at Mechanicsville, computed by Mr. R. P. Bloss, the engineer of the Duncan Company. A record is kept of the length and height of the flashboards at all times, with the dates of their setting and removal.

The flow over the dam has been computed by the Francis formula for the Merrimac dam, $Q=3.01\ LH^{1.53}$, L being 794 feet. The same formula has been used in all cases, whether flashboards are on or off.

The flow through the water wheels has been taken from the rating tables of the manufacturers. The working head on the wheels varies from 15 to 17 feet, depending on the condition of the flashboards on the dam. A test made by Mr. Bloss of a 39-inch Hercules wheel in the mill, which has been in use about eight years, shows the actual discharge to be substantially the same as that given in the manufacturers' tables when running at the speed of greatest efficiency. When running at higher speeds the discharge may be several per cent less.

A current-meter measurement of the flow below the dam was made at the Mechanicsville tollbridge October 20, and showed a discharge of 1,871 second-feet. The result is somewhat uncertain, however, owing to slack water. No water was flowing over the dam, and the calculated turbine discharge was 1,977 second-feet.

The flow of Hudson River at Mechanicsville has been calculated by George W. Rafter, member American Society of Civil Engineers, using the East Indian Engineers' formula for flow over dam.[1] This formula gives a somewhat greater discharge than that obtained by using the Francis formula for the Merrimac dam.

[1] Rept. State Engineer and Surveyor of New York, 1895, p. 105; also Water-Supply and Irrigation Paper U. S. Geol. Survey No. 24, p. 78

Daily discharge, in second-feet, of Hudson River at Mechanicsville, New York, for 1898.

[Drainage area, 4,500 square miles.]

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	6,252	4,340	5,011	20,306	*13,474	9,490	3,179	3,005	a4,345	3,222	7,954	6,330
2	(*)	4,839	4,998	18,016	11,545	8,159	3,014	3,888	a4,007	*2,877	7,667	6,130
3	5,319	5,034	4,927	*15,810	9,702	7,558	*3,007	6,225	a4,145	2,835	6,590	5,698
4	5,276	4,114	4,909	13,890	10,820	6,292	3,007	5,72[illegible]	a4,340	2,780	6,500	*5,195
5	5,563	3,956	4,791	11,932	10,570	*5,808	3,364	6,031	4,547	11,210	5,890	5,991
6	5,698	(*)	(*)	10,692	11,300	5,508	2,588	5,832	4,145	20,089	*5,803	6,300
7	6,873	4,000	4,998	9,928	11,032	4,729	2,453	*5,415	4,007	9,472	5,890	6,322
8	7,592	5,286	4,875	8,787	*10,250	4,399	2,314	5,091	4,930	8,261	5,503	5,456
9	(*)	5,139	6,418	8,158	9,700	4,249	2,220	4,335	4,217	*7,872	5,872	4,130
10	6,431	5,145	7,004	*7,924	8,695	3,978	*1,163	3,926	3,813	6,960	6,238	4,675
11	6,249	5,865	9,746	7,915	7,875	3,425	2,000	3,743	*3,368	6,038	20,256	*4,345
12	5,136	7,639	20,202	7,646	8,420	*3,857	2,182	3,387	3,304	6,109	16,202	4,290
13	11,930	(*)	(*)	8,553	11,054	4,299	2,058	3,331	3,188	5,331	*14,510	3,739
14	11,424	9,732	39,231	9,815	10,585	4,945	2,080	*7,148	2,732	4,829	13,202	3,540
15	10,557	8,078	36,155	9,824	*10,161	7,662	2,184	3,362	2,597	6,888	11,385	3,850
16	(*)	7,706	31,310	12,188	9,967	6,872	2,159	3,250	2,487	*7,338	10,366	4,016
17	10,176	5,396	31,069	*12,575	10,082	5,681	*1,163	3,223	2,597	8,125	9,722	4,495
18	7,898	6,978	31,794	12,188	10,637	4,620	2,192	3,235	*2,873	7,250	8,910	*4,590
19	7,318	7,224	29,304	11,155	9,404	*4,851	2,224	3,660	2,873	6,500	8,922	4,590
20	8,401	(*)	(*)	11,157	9,412	5,593	2,175	4,507	3,365	6,508	*10,505	4,885
21	14,053	7,445	35,917	10,922	8,355	4,603	2,192	*4,495	2,975	6,418	11,167	5,035
22	10,625	7,348	33,660	10,127	*7,954	4,399	3,188	4,718	2,899	6,310	10,366	5,035
23	(*)	6,746	30,004	9,815	8,000	4,198	3,565	3,927	2,666	*6,894	9,610	6,908
24	11,873	6,149	26,754	*9,627	8,400	4,348	*3,549	5,779	3,297	7,578	8,898	8,330
25	9,866	5,915	24,034	20,440	9,491	4,097	3,928	a11,167	*5,231	7,178	8,685	*7,747
26	9,173	5,461	21,040	24,210	11,953	*4,624	4,665	a9,953	6,878	6,693	7,847	6,886
27	8,171	(*)	(*)	24,338	14,758	4,115	4,258	a8,125	5,706	9,382	*6,413	5,872
28	8,101	5,380	19,077	20,440	15,327	3,556	3,656	*a8,250	4,775	11,042	5,830	4,618
29	7,727	.....	19,818	18,165	*14,140	3,508	3,352	a6,109	4,345	10,485	6,300	4,628
30	(*)	.....	22,777	14,873	12,727	3,238	3,175	a4,829	3,650	*10,059	6,300	4,829
31	4,822	.....	21,805	.....	10,490	.....	*3,015	a4,217	.....	9,953	.....	5,348
Mean	8,173	6,038	19,617	13,047	10,525	5,069	2,751	5,029	3,810	7,516	8,978	5,291

a Discharge August 25 to September 4 approximate, owing to irregular flashboards.
* Sunday.

Daily discharge, in second-feet, of Hudson River at Mechanicsville, New York, for 1899.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	*4,262	3,910	7,847	7,960	26,900	3,990	2,220	1,520	1,713	(*)	4,564	3,005
2	4,262	4,268	7,040	7,159	27,617	3,796	*1,047	629	1,561	4,042	8,899	3,414
3	5,209	4,268	6,403	6,955	25,551	3,601	2,128	1,213	(*)	3,453	12,020	*3,753
4	6,065	4,150	.....	7,938	22,472	*2,220	.....	1,153	1,293	2,850	11,899	5,545
5	9,146	*4,002	(*)	9,504	19,242	2,990	2,741	1,085	1,860	2,745	*10,346	6,6[illegible]
6	8,460	4,223	16,501	9,955	15,152	2,840	1,610	(*)	1,764	2,650	11,049	5,974
7	8,355	4,048	12,266	11,719	*12,662	2,728	1,670	1,494	1,706	2,571	9,308	5,210
8	*8,350	4,275	9,077	19,708	10,960	2,805	1,211	1,616	1,680	(*)	8,140	4,558
9	8,026	3,448	8,665	*18,275	9,270	2,525	(*)	1,475	1,466	2,590	7,166	3,805
10	6,562	3,665	7,909	15,062	8,587	2,290	3,000	484	(*)	2,712	6,662	*3,365
11	5,766	3,622	7,315	15,582	7,446	(*)	3,533	1,148	1,480	2,903	6,182	3,569
12	5,250	*4,062	*11,739	16,875	7,550	2,399	3,296	993	1,598	2,874	*5,490	4,979
13	5,272	4,196	15,437	17,145	7,155	2,505	3,243	(*)	1,471	2,926	6,321	11,1[illegible]
14	*5,960	4,508	12,153	21,378	*7,060	2,230	3,002	1,580	1,567	2,353	5,536	14,578
15	*7,435	4,600	10,555	27,975	7,772	2,135	2,800	1,680	1,484	(*)	5,179	14,979
16	8,910	4,798	10,012	*29,730	7,346	3,078	(*)	1,501	1,549	2,177	4,858	13,452
17	10,155	5,598	9,080	31,112	5,738	3,060	2,861	1,815	(*)	2,132	5,283	*9,646
18	9,685	5,691	7,895	29,667	5,729	*2,762	3,338	979	711	1,853	4,857	9,534
19	7,339	*5,620	*9,322	29,711	5,928	2,960	2,860	1,390	1,636	1,798	*4,359	9,9[illegible]
20	6,385	5,267	11,350	33,940	5,917	2,508	2,560	(*)	957	1,997	4,898	10,959
21	6,297	4,855	9,670	36,210	*5,925	2,372	2,485	1,610	941	2,185	4,729	11,0[illegible]
22	*5,770	5,686	9,337	36,210	6,187	2,220	2,295	1,123	1,448	(*)	4,587	10,722
23	5,697	10,083	8,645	*37,146	5,313	2,105	(*)	1,743	1,464	2,466	4,297	9,642
24	5,522	7,501	8,657	38,283	4,756	1,878	2,240	2,480	(*)	2,696	3,964	*8,0[illegible]
25	6,950	5,583	8,095	39,064	4,710	*304	2,238	1,383	1,516	2,372	3,731	8,241
26	5,775	*5,255	*7,492	41,475	4,462	1,878	2,188	1,463	2,116	2,304	*3,472	7,251
27	5,438	6,450	6,895	40,664	4,117	2,241	2,021	(*)	6,000	2,128	4,027	5,932
28	4,545	10,326	6,615	38,300	*3,985	2,290	2,140	1,756	5,127	2,216	3,495	5,635
29	*4,290	.....	7,835	33,908	4,125	2,368	1,686	1,738	4,413	(*)	3,439	4,[illegible]
30	4,345	.....	8,765	*30,292	3,787	2,512	(*)	1,739	4,870	3,198	3,318	4,075
31	4,066	.....	7,602	.....	3,918	.....	2,055	1,477	.....	3,815	.....	*2,56[illegible]
Mean	6,437	5,141	9,316	24,607	9,591	2,539	2,402	1,417	2,054	2,616	6,086	7,33[illegible]

* Sunday.

Daily discharge, in second-feet, of Hudson River at Mechanicsville, New York, for 1900.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	3,612	4,163	5,623	(*)	16,713	4,288	*2,364	2,977	2,365	2,102	2,531	9,107
2	3,276	3,349	12,924	10,088	15,493	4,932	2,368	2,809	*2,297	2,223	2,094	*8,316
3	3,294	3,737	10,299	10,413	14,676	*3,482	2,368	2,464	2,177	2,144	2,413	7,444
4	3,263	(*)	*3,800	11,188	14,593	7,644	2,191	2,386	2,519	2,071	*2,159	6,459
5	3,277	6,556	7,998	11,188	12,494	6,793	2,701	*2,167	2,289	1,752	1,977	10,694
6	3,602	8,140	5,932	12,267	*9,444	6,083	2,382	2,135	2,125	1,768	1,925	9,033
7	*3,000	7,204	9,312	15,149	9,741	5,554	2,109	2,266	2,031	*2,094	1,576	7,920
8	3,816	6,777	6,900	*17,520	10,924	5,329	*2,200	2,607	1,725	1,895	1,666	7,624
9	3,556	19,351	6,738	18,804	9,066	5,329	2,130	2,785	*1,248	1,888	2,028	*6,880
10	3,998	11,373	7,420	16,633	9,403	*4,430	2,007	2,752	713	1,965	2,520	6,181
11	3,698	*9,666	*6,342	14,742	10,716	5,827	1,437	2,182	1,738	1,692	*3,860	5,019
12	3,218	7,799	6,305	14,237	8,530	4,759	1,692	*2,550	1,916	2,063	4,331	4,663
13	3,159	24,539	5,563	14,362	*7,500	4,175	1,342	2,541	1,801	1,947	3,820	4,988
14	(*)	41,285	5,761	13,554	7,626	3,911	2,002	2,691	1,118	*1,931	3,805	4,856
15	3,067	25,354	5,363	*11,988	7,750	4,033	*2,263	3,292	1,433	2,123	3,315	4,250
16	3,158	22,593	4,411	13,588	9,390	4,054	2,292	4,299	*1,820	2,061	3,136	*4,250
17	3,180	19,527	4,719	15,918	7,549	*3,700	2,185	5,773	2,063	2,233	2,911	4,195
18	3,139	*16,547	*3,675	22,761	8,241	4,269	2,189	4,617	1,904	2,013	*3,036	4,265
19	4,668	14,173	4,935	33,150	7,500	3,617	1,991	*4,014	2,087	1,978	3,287	3,909
20	16,115	12,825	15,344	40,131	*9,450	2,941	1,580	3,955	2,032	1,954	3,829	4,129
21	*18,307	11,555	11,058	42,908	10,953	2,774	1,565	3,154	1,412	*1,977	5,292	3,953
22	11,307	10,060	7,999	*42,300	9,141	2,968	*2,180	2,834	1,581	2,017	9,332	4,004
23	10,176	13,310	8,324	43,546	8,029	3,141	2,299	2,510	*1,704	1,955	10,120	*4,210
24	9,853	10,188	9,399	41,940	7,354	*1,877	2,271	2,351	1,822	2,253	9,321	5,239
25	9,295	*9,301	*10,686	38,575	6,345	2,868	2,092	1,831	1,978	2,195	*8,796	3,763
26	8,449	7,042	8,226	33,983	5,758	3,210	2,690	*1,708	1,953	2,167	9,483	6,536
27	6,913	5,390	7,541	28,578	*4,651	3,713	4,082	1,702	2,002	2,203	11,027	4,700
28	(*)	5,252	7,626	25,618	6,454	2,787	3,898	1,814	2,422	*2,320	11,694	4,679
29	6,145	.....	7,626	*25,741	5,333	2,361	*2,836	2,192	2,145	2,583	11,347	4,459
30	5,808	.....	8,014	19,510	4,042	2,530	3,951	2,142	*2,135	3,287	9,670	*4,593
31	4,453	.....	9,075	.....	3,857	.....	3,248	2,278	.....	3,124	.....	4,535
Mean	5,841	12,484	7,740	22,614	8,962	4,093	2,352	2,703	1,886	2,128	5,077	5,331

* Sunday.

CROTON RIVER AT OLD CROTON DAM, NEW YORK.

Records of the yield of the Croton watershed have been prepared by the engineers of the aqueduct commission. Tables of the monthly discharge of the river, in second-feet, for the years 1870 to 1898, inclusive, were printed in the Twentieth Annual Report, Part IV, page 83, and in the Twenty-first Annual Report, Part IV, page 75, is a table of the rainfall and run-off of the watershed for the years 1868 to 1899, inclusive. The following table, furnished by Mr. W. R. Hill, chief engineer of the aqueduct commission, is the record for 1900. The figures of rainfall are those obtained at Boyds Corners, in Croton Basin.

Estimated monthly discharge of Croton River at old Croton dam, New York, for 1900.

[Drainage area, 338 square miles.]

Month.	Discharge.	Total.	Run-off.		Rainfall.
			Per square mile.	Depth.	
	Second-feet.	*Acre-feet.*	*Second-feet.*	*Inches.*	*Inches.*
January	537	33,020	1.59	1.83	4.18
February	2,179	121,032	6.45	6.71	7.97
March	1,724	106,013	5.10	5.88	5.06
April	553	32,915	1.64	1.83	2.16
May	716	43,991	2.12	2.45	6.40
June	357	21,254	1.06	1.18	2.17
July	78	4,774	.23	.27	4.28
August	77	4,716	.23	.27	1.75
September	107	6,357	.32	.36	3.27
October	174	10,674	.51	.59	4.73
November	346	20,573	1.02	1.14	4.91
December	639	39,258	1.89	2.18	2.58
The year	624	444,577	1.85	24.69	49.46

DELAWARE RIVER AT LAMBERTVILLE, NEW JERSEY.

This station, which was established July 23, 1897, by E. G. Paul, is described in Water-Supply Paper No. 35, page 62. The results of measurements for 1899 will be found in the Twenty-first Annual Report, Part IV, page 77. During 1900 the following measurements of discharge were made by Mr. Paul:

May 22: Gage height, 5.10 feet; discharge, 17,750 second-feet.
September 27: Gage height, 2.95 feet; discharge, 1,824 second-feet.

Daily gage height, in feet, of Delaware River at Lambertville, New Jersey, for 1900.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	(a)	(a)	5.40	4.25	4.45	3.30	2.65	3.40	2.95	2.90	2.80	4.15
2	(a)	(a)	12.00	4.20	4.35	3.40	2.50	3.15	3.05	2.50	2.75	4.05
3	(a)	(a)	10.70	4.30	4.25	3.45	2.60	3.00	2.75	2.55	2.85	3.95
4	(a)	7.00	7.95	4.40	4.20	3.65	2.90	3.05	2.90	2.65	3.20	3.85
5	(a)	6.85	6.80	4.50	4.05	3.80	3.10	3.05	2.70	2.80	3.25	4.40
6	(a)	5.75	6.20	4.40	4.00	3.70	3.40	2.70	2.70	2.55	3.20	4.90
7	(a)	5.30	6.05	4.30	3.95	3.55	3.80	3.25	2.60	2.75	3.20	5.00
8	(a)	5.45	5.80	4.40	3.90	3.35	3.65	2.70	2.65	2.70	3.15	4.95
9	(a)	5.60	6.00	4.60	3.80	3.30	3.65	2.60	2.90	2.75	3.20	4.80
10	(a)	6.95	5.60	4.55	3.80	3.55	3.55	2.65	2.85	2.75	3.10	4.65
11	(a)	6.55	5.45	4.45	3.90	3.85	3.50	2.80	3.00	2.75	2.90	4.20
12	5.65	5.60	5.20	4.30	3.80	3.80	3.35	2.55	3.25	2.60	2.75	3.95
13	4.35	7.46	4.90	4.35	3.80	3.50	3.35	2.55	3.10	2.50	2.55	3.90
14	3.95	11.40	4.60	4.40	3.80	3.45	3.20	3.05	3.00	3.15	2.70	4.00
15	3.70	9.70	4.55	4.50	3.70	3.95	3.30	3.05	2.85	2.95	2.60	(a)
16	3.60	8.10	4.50	4.45	3.60	3.95	3.10	2.80	3.45	3.00	2.85	(a)
17	3.85	6.65	4.20	4.30	3.60	3.85	3.35	2.85	3.25	2.80	2.85	(a)
18	3.80	5.95	4.20	4.75	4.00	3.70	3.25	2.80	3.00	2.60	3.10	(a)
19	3.70	5.25	4.10	6.60	4.80	3.45	3.30	2.95	3.00	2.40	2.85	(a)
20	4.45	5.05	5.15	7.15	5.35	3.45	3.35	3.15	2.70	2.35	2.80	(a)
21	7.70	4.90	5.90	6.50	5.40	3.25	3.30	3.10	2.55	2.60	2.75	(a)
22	7.75	6.05	6.30	6.00	5.05	3.20	3.50	2.95	2.65	2.65	2.85	(a)
23	6.20	9.65	5.65	5.85	4.60	3.05	3.35	2.90	2.85	2.25	3.10	(a)
24	5.75	8.70	5.30	5.95	4.25	3.15	3.45	2.80	2.85	2.40	3.40	3.90
25	5.25	7.10	5.15	5.95	4.15	3.20	3.35	2.65	2.80	2.40	3.15	3.60
26	5.00	6.10	5.00	5.70	4.05	3.15	4.35	3.10	2.90	2.50	3.75	3.50
27	5.20	5.25	4.85	5.25	3.95	3.00	4.10	3.05	2.95	2.50	3.50	3.90
28	(a)	4.90	4.95	4.95	3.75	2.90	3.85	2.95	2.90	2.85	4.10	4.15
29	(a)	.....	4.60	4.80	3.60	2.70	3.65	2.80	2.65	2.80	4.75	4.05
30	(a)	.....	4.50	4.65	3.60	2.70	3.60	2.85	2.95	2.85	4.40	(a)
31	(a)	.....	4.35	.....	3.45	.....	3.50	3.10	.....	2.95	.....	4.00

a Frozen.

TOHICKON CREEK AT POINT PLEASANT, PENNSYLVANIA.

The stream is described in the Twentieth Annual Report, Part IV, page 98, which also contains (pages 98–103) the figures for monthly flow for the years 1890 to 1898. Figures for daily flow for 1899 were published in Water-Supply Paper No. 35, page 64. In the Twenty-first Annual Report, Part IV, pages 83 to 85, will be found the monthly flow for 1899 and diagrams of the discharge from 1883 to 1899, inclusive. The following tables give the figures for daily flow since 1883, when the station was established, from which the diagrams were constructed. Daily records of gage heights were not kept during 1900. The average monthly discharge, which will appear in the Twenty-second Annual Report, Part IV, has been computed by Mr. John E. Codman, hydrographer of the water department of the city of Philadelphia, from the rainfall and the monthly flow of Neshaminy and Perkiomen creeks.

Daily discharge, in second-feet, of Tohickon Creek at Point Pleasant, Pennsylvania, for 1883.

[Drainage area, 102.2 square miles.]

Day.	Sept.	Oct.	Nov.	Dec.	Day.	Sept.	Oct.	Nov.	Dec.
1	.....	3	90	49	18	11	7	20	41
2	.....	6	55	46	19	5	9	15	41
3	.....	10	39	43	20	4	18	23	41
4	.....	28	33	31	21	3	12	17	110
5	.....	24	26	43	22	4	12	21	100
6	.....	11	26	37	23	2	13	18	100
7	.....	11	30	41	24	2	456	18	100
8	.....	11	30	61	25	5	189	21	100
9	.....	13	36	81	26	5	99	24	100
10	.....	7	35	122	27	8	95	220	100
11	.....	12	151	81	28	6	69	165	260
12	.....	7	118	41	29	5	356	85	250
13	4	9	79	41	30	5	687	49	240
14	7	5	54	41	31	.....	179	.....	230
15	7	7	43	41					
16	1	18	29	41	Mean	5	77	53	86
17	2	15	20	41					

Daily discharge, in second-feet, of Tohickon Creek at Point Pleasant, Pennsylvania, for 1884.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	331	406	384	359	29	10	589	35	29	7	150	139
2	473	395	376	333	26	10	372	41	18	6	125	139
3	515	395	368	400	20	8	247	30	18	5	102	139
4	445	396	360	279	20	6	216	52	18	5	205	46
5	354	396	419	217	20	5	2,309	100	11	5	313	44
6	354	396	392	155	20	7	1,507	103	5	4	253	256
7	467	1,989	365	92	19	8	863	76	7	2	81	1,560
8	2,593	2,412	362	80	18	9	612	42	7	2	64	319
9	3,492	1,270	359	406	17	10	256	32	7	2	21	164
10	2,749	1,270	356	378	129	35	131	22	7	2	21	77
11	1,579	889	1,113	338	150	35	39	20	9	1	22	50
12	590	399	1,871	305	103	34	25	20	6	1	23	662
13	590	1,743	1,148	223	75	33	31	19	2	1	19	651
14	502	2,103	425	141	349	33	31	11	2	1	12	173
15	390	585	410	160	204	33	31	10	3	1	12	1,112
16	390	434	395	300	93	33	31	8	3	1	12	367
17	390	621	380	258	54	33	26	5	3	0	12	159
18	377	1,218	365	216	36	34	22	5	3	0	12	104
19	361	1,000	1,114	173	34	34	22	4	4	0	10	73
20	361	1,862	1,390	109	49	33	15	4	4	0	13	251
21	382	1,314	414	46	67	31	15	4	2	0	25	315
22	409	1,204	400	42	59	22	15	7	3	0	25	223
23	409	2,008	380	38	43	9	10	12	3	2	46	365
24	409	900	1,000	44	29	4	9	11	4	4	859	211
25	409	855	425	35	24	4	9	9	5	5	211	159
26	409	739	800	28	22	4,379	7	4	4	4	106	123
27	409	623	568	21	15	1,460	5	3	2	5	68	100
28	411	507	336	15	13	1,242	7	4	2	8	52	63
29	415	392	321	15	10	1,024	11	17	2	20	201	107
30	415	.....	306	41	10	807	20	26	6	20	159	240
31	415	.....	385	.....	9	.....	22	28	.....	250	.....	2,172
Mean	703	990	580	175	57	314	242	25	7	12	108	353

Daily discharge, in second-feet, of Tohickon Creek at Point Pleasant, Pennsylvania, for 1885.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	2,614	75	59	1,291	50	8	1	1	8	1	84	13
2	480	73	61	1,004	62	12	2	35	7	5	685	10
3	138	72	102	518	60	15	3	696	7	3	345	164
4	138	75	119	1,013	60	10	3	1,439	7	5	131	135
5	140	58	183	3,664	55	13	109	188	2	5	73	12
6	1,599	79	218	2,411	28	12	325	50	5	3	61	12
7	949	116	231	868	37	9	107	58	5	6	55	8
8	232	135	196	203	172	6	14	210	4	7	953	12
9	176	425	174	169	186	9	5	180	6	6	1,137	172
10	167	1,463	115	146	82	10	5	120	4	5	286	50
11	82	1,885	83	118	63	10	4	54	6	10	151	5
12	1,404	738	69	118	63	9	4	29	6	3	94	17
13	423	403	59	112	63	4	3	33	7	18	69	32
14	103	283	64	76	61	2	3	61	4	12	57	8
15	293	477	149	76	29	4	3	49	1	14	41	36
16	784	697	162	76	25	6	3	29	3	12	46	14
17	1,267	724	202	76	23	8	1	25	5	11	39	14
18	216	238	180	59	23	3	1	19	2	10	35	15
19	101	218	178	42	22	7	1	17	1	6	28	1
20	94	202	170	42	16	6	1	7	1	8	33	7
21	87	192	116	40	14	6	1	11	5	44	29	4
22	91	177	69	29	14	5	0	18	1	112	23	6
23	98	163	64	29	14	4	0	17	1	66	244	5
24	101	150	51	29	19	4	1	17	4	34	720	5
25	104	116	190	29	20	1	1	14	2	17	449	5
26	97	107	592	36	25	2	1	10	2	24	363	5
27	82	90	931	39	18	4	1	8	1	27	323	6
28	90	86	1,027	43	20	5	1	13	1	24	235	8
29	96	.....	783	49	16	4	1	12	1	12	194	9
30	99	.....	783	49	15	2	1	5	1	244	147	3
31	89	.....	850	.....	12	.....	1	8	.....	302	.....	6
Mean	401	340	265	415	44	7	20	112	3	31	236	57

Daily discharge, in second-feet, of Tohickon Creek at Point Pleasant, Pennsylvania, for 1886.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	278	172	169	2,523	32	29	24	17	3	2	15	62
2	112	137	159	647	21	25	19	21	1	4	12	46
3	83	113	149	198	17	27	20	18	5	4	14	59
4	833	146	139	149	26	36	10	12	2	3	12	110
5	3,352	138	130	222	23	37	17	11	1	1	8	104
6	720	132	95	3,692	26	24	15	9	2	1	13	63
7	148	126	55	2,320	24	28	12	15	1	2	9	88
8	113	120	50	1,085	2,121	19	10	18	1	3	9	68
9	105	114	44	428	1,746	18	9	22	3	2	7	63
10	182	95	42	239	348	22	14	16	2	1	9	86
11	207	4,060	41	160	159	42	19	11	1	1	10	55
12	258	5,359	38	150	117	32	21	7	5	3	9	23
13	236	5,404	46	285	649	25	19	8	4	2	28	34
14	176	2,299	42	283	1,319	176	16	10	5	1	30	252
15	110	1,392	45	130	313	482	16	7	5	2	33	283
16	149	836	41	85	673	181	256	5	2	2	28	156
17	150	207	38	72	283	95	378	6	2	1	21	225
18	150	142	35	52	136	72	132	6	2	2	166	372
19	227	149	43	50	116	39	275	7	2	1	413	1,016
20	372	136	46	43	125	26	83	6	2	5	96	710
21	110	107	2,061	43	140	26	51	3	5	1	42	230
22	171	140	1,088	32	117	22	52	4	5	1	31	148
23	223	91	308	26	237	1,030	40	3	3	2	446	99
24	163	87	146	31	140	825	26	4	2	1	621	148
25	311	1,123	125	37	115	210	20	3	1	2	756	985
26	197	2,108	101	35	94	128	366	6	1	4	2,064	336
27	117	324	100	35	77	81	100	6	2	8	257	189
28	679	231	783	34	81	54	36	8	1	15	107	103
29	984	.....	948	32	67	38	28	7	3	17	73	142
30	908	.....	2,150	24	43	28	25	3	1	12	61	144
31	200	.....	2,598	.....	41	.....	25	2	.....	15	.....	175
Mean	388	910	382	438	304	129	69	9	2	4	180	212

Daily discharge, in second-feet, of Tohickon Creek at Point Pleasant, Pennsylvania, for 1887.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	330	205	246	106	57	188	18	1,148	20	18	24	25
2	400	109	155	101	49	161	12	196	18	20	18	33
3	300	107	152	132	39	148	10	279	16	25	16	24
4	220	127	121	153	38	90	11	86	14	26	18	24
5	150	155	96	142	36	50	434	52	15	22	14	20
6	180	136	86	101	29	40	1,858	85	15	22	15	24
7	130	438	180	73	244	35	419	116	13	14	9	35
8	120	884	816	61	669	44	149	68	10	16	14	35
9	100	838	736	54	508	38	82	46	11	11	15	30
10	110	531	1,149	44	195	34	79	24	11	14	13	70
11	130	770	612	49	118	25	68	22	10	14	20	1,201
12	115	623	210	46	73	17	43	39	180	13	18	639
13	110	275	159	40	58	23	35	39	349	14	18	470
14	720	168	145	33	43	20	23	22	111	13	16	114
15	744	425	109	33	31	14	15	24	59	11	22	428
16	416	644	83	33	37	11	14	18	41	9	20	708
17	252	329	66	24	34	14	70	21	31	7	20	205
18	236	1,581	51	40	26	13	63	19	18	11	16	75
19	145	2,357	50	162	20	9	40	16	21	10	22	131
20	113	463	48	312	20	25	29	22	15	10	47	137
21	110	252	59	131	18	27	22	19	19	90	79	184
22	175	332	125	83	15	23	19	23	15	75	49	291
23	523	375	275	67	20	1,483	78	166	17	54	35	320
24	2,544	641	284	113	31	446	335	1,883	14	35	28	234
25	1,112	466	945	96	45	139	284	549	10	25	27	177
26	602	166	884	134	32	63	106	151	12	22	27	187
27	214	613	351	147	24	46	57	89	14	19	19	158
28	202	466	1,538	88	17	30	42	47	19	17	21	701
29	1,473	.....	540	100	31	25	30	36	25	22	27	1,139
30	1,542	.....	176	92	19	23	26	27	20	14	30	652
31	415	.....	144	.....	.....	.....	36	23	.....	21	.....	306
Mean	449	517	342	93	87	110	145	173	38	23	24	283

Daily discharge, in second-feet, of Tohickon Creek at Point Pleasant, Pennsylvania, for 1888.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	3,190	150	122	318	37	43	11	8	21	38	94	144
2	3,492	150	120	235	32	33	7	8	18	40	80	394
3	683	116	130	189	31	22	9	7	20	37	76	275
4	294	109	164	131	28	27	4	11	17	35	71	81
5	217	201	154	692	28	27	11	16	21	32	65	81
6	154	446	125	2,019	28	22	3	22	19	72	60	83
7	198	712	106	1,790	29	25	3	14	9	265	51	87
8	669	671	85	911	31	17	7	47	10	199	49	83
9	514	608	83	135	33	12	4	48	25	99	67	172
10	237	521	79	780	34	14	8	11	111	47	336	282
11	143	401	100	1,797	39	13	8	11	199	36	494	159
12	151	253	908	435	89	20	6	26	604	38	173	115
13	188	208	123	237	92	22	11	34	337	41	98	84
14	280	190	750	382	78	27	7	29	165	42	81	71
15	234	122	473	344	70	17	3	23	87	42	1,130	71
16	334	104	380	254	63	6	3	15	103	43	982	63
17	606	118	570	291	51	6	8	18	1,351	43	333	2,374
18	714	127	620	150	45	4	4	15	5,546	39	237	2,069
19	647	352	494	113	45	8	8	11	1,991	39	935	434
20	590	1,377	950	103	52	8	2	4	270	53	864	148
21	590	2,768	2,659	86	47	9	2	392	2,710	62	311	133
22	369	1,751	1,886	63	36	12	8	3,499	472	56	175	176
23	350	890	504	61	34	9	7	333	191	48	131	265
24	310	828	253	52	35	6	8	92	117	82	122	117
25	185	2,188	151	44	39	8	5	52	84	110	132	72
26	226	1,650	173	41	38	8	5	30	75	82	214	72
27	242	661	1,633	38	35	3	6	25	73	491	423	245
28	405	526	938	33	37	7	11	24	60	993	401	337
29	120	277	1,330	32	63	10	10	19	41	694	212	194
30	86	.....	630	36	78	13	9	15	36	229	169	111
31	289	.....	460	.....	57	.....	9	17	.....	131	.....	91
Mean	571	637	556	393	46	15	7	157	499	138	286	330

Daily discharge, in second-feet, of Tohickon Creek at Point Pleasant, Pennsylvania, for 1889.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	86	124	36	295	135	340	210	2,602	17	63	139	151
2	87	99	52	958	101	441	999	815	16	68	131	139
3	77	92	386	330	79	156	1,482	367	17	62	1,041	111
4	72	86	2,567	183	71	82	1,053	195	15	49	813	105
5	124	82	2,141	133	63	64	697	211	20	46	263	95
6	968	71	864	108	54	52	574	194	11	42	166	75
7	637	89	320	83	48	45	123	119	9	35	131	75
8	263	125	195	78	43	39	80	84	11	33	116	136
9	691	148	134	75	42	32	54	66	11	31	3,304	351
10	1,080	88	108	66	36	31	38	62	14	30	2,308	214
11	288	61	100	55	43	31	70	55	16	32	406	165
12	149	60	84	53	40	242	111	51	16	55	223	165
13	104	60	79	64	40	308	77	83	44	96	963	111
14	99	61	92	67	44	63	141	2,115	160	314	1,339	165
15	87	45	97	64	44	110	2,156	2,177	155	248	331	59
16	81	70	84	61	38	565	787	406	532	111	166	177
17	1,530	514	75	56	37	263	188	164	2,019	68	119	590
18	742	819	88	54	34	383	82	96	3,028	51	102	66
19	230	663	105	49	16	161	262	69	920	44	1,623	33
20	112	228	113	49	363	82	1,975	59	367	43	1,041	24
21	418	143	214	44	911	50	422	49	201	97	508	15
22	397	94	951	42	450	39	139	41	163	129	816	179
23	263	66	623	71	186	33	88	37	115	104	302	136
24	211	51	266	54	95	28	69	39	85	97	163	92
25	258	74	158	29	68	29	48	33	459	72	457	31
26	313	73	114	424	68	807	41	35	524	57	557	15
27	778	45	97	2,126	357	610	58	37	260	2,958	734	135
28	1,096	37	112	1,439	744	214	65	33	140	1,643	2,768	14
29	440	.....	124	582	244	368	91	22	98	392	645	3
30	187	.....	104	241	118	215	844	19	70	194	234	6
31	191	.....	87	.....	80	.....	4,714	20	.....	147	.....	3
Mean	389	149	341	264	151	189	570	334	317	207	733	72

Daily discharge, in second-feet, of Tohickon Creek at Point Pleasant, Pennsylvania, for 1890.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	51	114	766	176	41	51	14	22	34	23	75	28
2	53	84	349	197	34	50	125	17	30	18	64	35
3	57	95	124	126	34	33	436	12	27	810	52	62
4	51	155	92	378	24	33	687	15	21	783	48	87
5	62	239	87	721	461	37	147	18	17	194	48	87
6	98	131	83	191	282	38	71	53	18	96	42	74
7	102	75	71	143	297	107	47	32	12	294	38	74
8	87	2,512	71	182	153	66	36	27	14	281	37	140
9	76	2,000	64	778	115	41	45	25	12	163	35	134
10	57	293	71	758	78	32	23	21	17	91	37	60
11	48	176	563	241	60	22	18	16	10	67	45	44
12	45	141	978	128	53	91	12	22	15	49	107	45
13	49	117	440	99	47	485	12	19	97	25	89	142
14	45	420	1,182	84	117	352	12	14	438	54	63	141
15	1,110	806	1,993	68	1,037	118	16	13	612	75	48	51
16	1,586	192	588	61	480	65	18	11	797	73	43	64
17	422	124	228	54	146	46	19	10	624	965	112	243
18	160	107	166	48	88	38	20	14	179	405	249	1,044
19	108	94	132	48	65	36	17	27	81	143	100	239
20	169	696	151	42	214	31	12	478	43	118	89	154
21	285	414	1,290	36	292	31	12	290	32	104	64	121
22	109	146	2,942	42	117	34	11	130	32	79	54	90
23	120	107	1,908	44	70	35	4	159	32	469	85	88
24	109	115	422	39	53	44	9	57	22	2,624	82	68
25	78	186	214	36	42	42	11	38	24	944	41	80
26	50	219	436	42	1,278	27	317	36	24	358	31	173
27	62	154	264	51	1,967	20	122	405	27	183	26	198
28	71	482	1,695	56	416	17	68	261	23	122	21	191
29	64	.....	919	50	154	15	43	95	23	103	33	106
30	97	.....	206	43	92	25	34	119	22	128	27	73
31	160	.....	161	.....	67	.....	29	45	.....	115	.....	68
Mean	181	371	602	165	270	68	79	81	112	320	63	135

Daily discharge, in second-feet of Tohickon Creek at Point Pleasant, Pennsylvania, for 1891.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	61	1,726	202	125	24	19	8	172	276	10	27	56
2	1,397	1,105	148	178	20	18	7	91	123	11	30	38
3	1,421	779	110	814	21	13	8	60	92	10	20	33
4	243	614	85	406	61	11	7	38	78	8	17	681
5	150	213	93	178	51	12	7	31	326	15	17	1,418
6	116	180	93	125	40	13	6	31	598	15	18	330
7	148	210	82	104	34	12	6	31	249	11	21	915
8	104	637	116	91	33	14	5	31	116	20	16	464
9	85	516	713	77	29	26	13	23	68	16	7	231
10	57	972	1,500	68	22	24	9	21	53	10	16	129
11	767	386	601	118	25	15	12	16	49	9	20	99
12	2,331	204	718	797	25	13	5	19	49	11	29	82
13	354	152	1,184	273	15	12	4	16	42	18	27	73
14	179	130	627	133	18	7	7	14	36	12	29	69
15	127	97	238	102	18	14	3	43	30	10	24	63
16	94	975	158	82	24	12	7	47	23	10	21	60
17	185	782	228	75	20	26	3	54	26	7	58	63
18	748	820	113	73	27	15	34	587	31	18	136	74
19	267	663	97	62	29	16	299	672	24	5	98	71
20	188	241	255	51	27	35	120	167	67	95	61	46
21	141	751	2,858	45	21	23	62	77	67	70	46	39
22	2,451	1,071	1,219	45	23	30	41	57	18	127	42	41
23	1,146	383	1,004	45	18	28	28	214	18	212	128	410
24	364	190	506	51	16	23	29	3,159	18	100	386	1,670
25	250	248	241	49	12	22	31	2,700	13	52	108	1,400
26	250	418	158	38	20	14	25	516	13	86	84	668
27	234	405	119	32	19	21	23	237	13	93	75	819
28	204	272	105	28	21	17	17	596	15	49	65	291
29	466	.....	95	28	15	6	571	357	15	39	54	184
30	1,269	.....	79	27	20	9	611	345	7	33	59	887
31	1,091	.....	79	.....	16	.....	515	415	.....	29	.....	351
Mean	545	540	446	144	25	17	81	349	85	39	58	379

Daily discharge, in second-feet, of Tohickon Creek at Point Pleasant, Pennsylvania, for 1892.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	148	157	1,645	113	40	76	59	21	20	15	8	190
2	1,090	145	831	100	47	60	46	37	16	8	9	9[illegible]
3	1,177	205	433	100	159	43	249	31	12	11	8	71
4	262	237	348	105	108	45	455	33	10	14	6	6[illegible]
5	158	180	283	95	70	64	164	21	15	17	9	59
6	108	140	218	86	60	73	78	24	19	14	9	5[illegible]
7	120	134	237	73	55	58	44	9	13	7	6	49
8	156	119	1,421	61	57	43	31	15	13	8	7	15[illegible]
9	150	175	1,447	57	51	90	26	15	16	7	8	20[illegible]
10	122	129	625	47	43	555	20	15	12	9	57	11[illegible]
11	100	102	390	49	39	287	19	16	5	11	301	16[illegible]
12	79	57	207	37	42	108	18	57	9	14	113	78
13	2,863	71	130	35	39	61	16	101	10	11	64	54
14	3,158	79	97	33	39	46	17	38	13	6	48	23[illegible]
15	2,580	72	84	133	143	41	14	23	17	8	803	62[illegible]
16	660	77	68	177	125	32	12	23	13	8	2,962	30[illegible]
17	260	86	66	90	86	31	10	13	15	6	522	15[illegible]
18	261	74	86	57	57	29	11	14	12	8	669	119
19	1,618	53	87	42	55	17	12	11	16	6	896	97
20	880	35	65	35	340	22	7	9	17	7	509	8[illegible]
21	352	73	65	44	401	23	6	11	27	4	370	15
22	161	109	65	95	529	15	5	10	11	8	88	7[illegible]
23	146	112	65	191	458	15	8	10	18	5	64	15
24	131	105	271	131	260	17	9	8	25	4	51	12[illegible]
25	121	92	727	71	177	16	8	9	13	4	48	18[illegible]
26	119	156	574	57	132	20	8	12	227	6	40	24[illegible]
27	129	190	1,036	46	1,333	26	5	80	59	11	35	15[illegible]
28	193	121	1,115	46	522	21	5	92	33	10	180	30[illegible]
29	277	87	413	50	184	14	8	75	22	6	676	18[illegible]
30	226	.....	194	50	112	14	9	30	19	5	195	10[illegible]
31	166	.....	135	.....	86	.....	12	30	.....	5	.....	6[illegible]
Mean	580	116	433	77	188	65	45	29	24	8	292	14[illegible]

Daily discharge, in second-feet, of Tohickon Creek at Point Pleasant, Pennsylvania, for 1893.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	474	489	78	78	279	35	16	8	49	11	51	12[illegible]
2	1,406	803	97	68	348	33	15	12	41	11	48	1[illegible]
3	301	1,366	108	51	832	21	15	9	32	15	48	19[illegible]
4	158	1,118	108	58	2,994	20	25	4	25	12	1,094	1,[illegible]
5	104	948	108	50	1,102	19	12	3	21	10	1,288	[illegible]
6	100	1,166	103	41	872	37	13	9	17	12	489	1[illegible]
7	131	1,113	87	49	413	95	12	5	22	17	184	17[illegible]
8	140	742	103	140	207	53	12	4	35	12	123	14[illegible]
9	140	320	965	145	124	39	9	10	21	10	101	13[illegible]
10	160	2,597	2,200	132	89	37	9	5	13	12	79	14[illegible]
11	204	1,735	800	249	81	32	11	4	18	11	71	25[illegible]
12	160	437	2,400	156	71	21	9	6	23	11	62	1[illegible]
13	236	462	856	106	62	37	6	5	13	12	54	[illegible]
14	203	846	575	358	58	14	6	6	15	30	60	13[illegible]
15	138	905	600	1,687	65	13	12	3	26	47	103	1[illegible]
16	165	1,113	380	701	330	17	12	3	834	49	130	1,3[illegible]
17	170	672	251	238	919	16	12	3	531	39	90	1,3[illegible]
18	118	248	191	165	613	15	13	3	131	31	60	3[illegible]
19	95	237	165	135	301	13	9	4	75	22	53	17[illegible]
20	63	201	162	515	87	9	7	345	55	20	48	1[illegible]
21	64	162	197	1,903	71	9	5	409	55	23	46	11[illegible]
22	72	136	246	448	57	170	6	79	54	15	496	[illegible]
23	105	114	266	185	54	217	6	36	34	30	312	[illegible]
24	93	92	295	135	49	58	4	1,814	16	68	120	[illegible]
25	61	87	318	102	46	25	3	503	25	59	81	1[illegible]
26	59	87	254	97	42	42	3	119	27	46	62	1[illegible]
27	65	87	158	237	40	38	4	58	17	79	55	[illegible]
28	53	73	114	289	33	25	7	44	18	506	1,244	[illegible]
29	83	.....	97	100	31	21	9	499	20	279	479	[illegible]
30	312	.....	92	118	33	21	7	198	17	116	188	[illegible]
31	511	.....	83	.....	33	.....	4	92	.....	66	.....	[illegible]
Mean	197	655	402	293	333	40	10	139	76	55	244	[illegible]

Daily discharge, in second-feet, of Tohickon Creek at Point Pleasant, Pennsylvania, for 1894.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	73	125	216	64	47	267	20	7	4	13	522	37
2	76	113	148	62	41	189	16	6	4	16	189	242
3	75	89	297	62	35	142	12	8	3	11	1,193	430
4	61	65	790	54	33	125	10	25	3	12	707	163
5	74	88	1,065	254	34	91	12	30	2	13	719	97
6	114	119	1,174	141	34	75	29	24	2	11	966	73
7	131	156	1,102	92	58	64	47	35	3	7	282	58
8	109	115	488	79	53	61	31	18	4	9	545	126
9	87	651	258	71	50	49	22	8	5	12	439	606
10	70	1,113	220	73	42	40	17	5	20	1,674	224	284
11	52	911	188	73	28	34	16	5	31	885	168	635
12	46	383	153	128	29	27	11	5	33	127	115	3,342
13	46	140	131	1,168	16	23	7	5	25	125	83	1,811
14	49	140	109	1,283	17	23	10	6	23	186	71	377
15	49	119	92	800	19	20	7	6	20	116	61	195
16	64	107	102	223	17	19	3	8	21	68	65	143
17	64	113	93	144	17	15	6	8	23	47	79	119
18	51	928	71	103	16	15	10	11	3,510	39	104	97
19	51	1,606	67	79	17	14	8	11	3,556	34	104	83
20	46	1,257	64	71	616	10	6	8	1,306	32	73	75
21	41	723	61	71	8,650	12	11	8	245	27	73	66
22	43	379	71	303	4,310	7	68	9	83	23	97	61
23	45	208	534	321	904	7	49	9	50	21	93	58
24	51	118	351	132	1,827	6	25	9	41	29	75	57
25	89	151	143	101	1,577	13	21	10	29	725	63	46
26	98	151	122	81	450	36	14	6	23	477	61	42
27	75	151	100	68	228	29	10	6	19	168	49	44
28	68	231	76	70	1,386	26	8	11	20	99	44	83
29	68	.....	70	58	2,252	19	5	4	36	67	46	109
30	107	.....	79	47	506	19	4	4	28	52	39	143
31	137	.....	79	.....	267	.....	6	4	.....	649	.....	113
Mean	71	373	275	209	760	49	17	10	306	186	245	316

Daily discharge, in second-feet, of Tohickon Creek at Point Pleasant, Pennsylvania, for 1895.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	62	138	1,296	122	107	36	61	7	4	3	15	22
2	62	156	1,064	117	86	33	96	6	3	3	23	54
3	80	180	706	210	157	39	61	8	5	3	23	176
4	89	163	495	239	47	32	57	310	6	3	29	86
5	127	119	352	131	42	26	579	145	2	2	16	51
6	50	193	248	99	39	57	648	29	4	3	22	38
7	381	104	197	89	40	58	240	16	5	3	21	25
8	628	71	1,116	1,404	37	47	100	9	6	3	17	20
9	418	54	943	3,857	35	26	54	12	4	4	12	18
10	498	92	494	1,760	35	29	35	10	3	3	10	20
11	2,942	168	451	616	31	23	27	9	3	3	9	18
12	1,109	168	317	146	25	20	21	229	3	5	15	14
13	262	108	383	1,131	29	26	18	41	3	15	13	17
14	204	92	1,519	1,191	39	22	12	27	3	26	12	11
15	146	70	437	528	155	12	15	25	3	27	13	8
16	146	51	200	271	93	10	18	17	3	25	12	6
17	169	40	166	117	63	9	12	17	3	19	11	7
18	169	35	192	88	44	10	11	8	3	14	15	7
19	112	49	343	75	31	11	10	6	3	14	22	7
20	80	58	430	62	33	11	8	10	3	11	15	6
21	88	148	328	51	33	9	8	9	3	5	15	14
22	79	246	328	49	35	10	25	5	3	7	11	420
23	89	255	362	47	32	8	20	7	3	8	12	242
24	107	255	334	43	33	7	14	6	3	7	9	91
25	107	245	525	41	34	9	12	5	3	6	9	54
26	1,632	245	576	41	23	14	11	5	3	6	16	46
27	515	266	299	42	31	16	11	4	3	6	63	60
28	184	902	229	59	351	41	9	4	3	5	78	68
29	141	.....	189	76	120	45	7	5	3	5	38	57
30	97	.....	132	92	53	50	11	4	3	5	27	64
31	102	.....	114	.....	50	.....	7	4	.....	8	.....	109
Mean	351	167	476	426	62	25	71	32	3	8	20	59

Daily discharge, in second-feet, of Tohickon Creek at **Point Pleasant**, *Pennsylvania, for 1896.*

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	327	55	498	297	20	35	11	113	3	167	23	177
2	98	90	192	236	23	31	10	56	3	59	19	104
3	59	83	50	193	18	28	10	35	4	27	20	73
4	39	58	48	101	21	18	7	27	5	27	22	65
5	39	138	59	71	26	10	5	17	10	29	1,851	59
6	28	6,515	59	72	25	12	13	22	1,665	24	1,249	51
7	17	1,720	57	81	25	11	16	12	319	20	273	61
8	16	487	44	85	19	9	38	12	95	17	180	72
9	13	201	42	72	13	12	13	10	44	10	279	239
10	9	174	44	57	11	11	104	9	29	16	131	242
11	7	225	35	53	12	9	76	12	23	16	109	98
12	7	156	40	51	13	7	34	9	14	29	123	71
13	8	91	51	11	4	4	22	10	10	1,130	136	59
14	13	180	51	51	11	9	15	47	10	430	108	56
15	13	327	41	40	12	28	10	16	10	215	73	53
16	6	206	71	33	11	20	12	7	16	108	55	44
17	6	107	77	33	7	24	10	8	21	68	47	52
18	6	90	54	36	7	28	9	8	328	48	44	57
19	5	85	2,028	25	11	23	6	8	112	38	46	44
20	5	75	3,343	21	12	17	6	9	49	28	42	49
21	5	75	1,205	40	10	8	9	7	35	29	36	51
22	4	70	742	49	14	14	2,417	5	23	33	36	42
23	6	62	449	40	20	16	704	3	19	28	36	44
24	151	47	193	31	16	11	304	6	20	55	35	44
25	239	51	98	35	17	15	87	7	13	76	37	35
26	136	69	371	39	23	21	46	6	11	54	42	39
27	70	88	596	39	18	18	47	5	8	38	35	39
28	59	229	311	35	48	10	163	5	8	31	45	39
29	46	851	2,444	26	137	13	184	5	14	31	856	35
30	28	.....	1,238	20	143	15	2,182	3	156	30	439	31
31	31	.....	527	.....	97	.....	410	3	.....	27	.....	32
Mean	48	435	486	67	27	16	225	16	102	94	214	71

Daily discharge, in second-feet, of Tohickon Creek at **Point Pleasant**, *Pennsylvania, for 1897.*

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	31	73	58	46	20	70	10	64	17	3	6	[illegible]
2	49	97	65	40	2,587	40	9	49	14	3	1,400	[illegible]
3	157	79	54	31	954	44	7	43	31	3	616	[illegible]
4	330	65	75	15	376	66	6	38	42	4	131	[illegible]
5	404	49	142	42	207	76	6	103	21	4	63	1,765
6	256	430	523	70	114	71	5	75	17	5	44	[illegible]
7	89	2,312	296	71	80	52	4	110	14	5	34	[illegible]
8	71	882	119	56	60	61	5	84	11	6	29	[illegible]
9	65	311	102	1,454	47	2,389	6	33	11	7	41	[illegible]
10	53	175	135	957	58	906	6	28	11	3	63	[illegible]
11	49	109	152	227	60	240	5	19	10	3	61	[illegible]
12	49	68	191	119	57	111	4	22	10	3	60	[illegible]
13	49	78	210	93	3,683	75	6	20	5	3	60	[illegible]
14	49	89	150	75	1,967	53	35	15	6	3	42	1,[illegible]
15	44	65	147	101	973	50	35	8	7	8	37	3,[illegible]
16	31	73	135	160	354	48	22	50	7	9	42	[illegible]
17	23	246	89	137	144	46	14	36	7	5	50	[illegible]
18	80	404	79	109	90	49	879	34	6	3	64	[illegible]
19	117	320	220	74	74	44	230	32	4	6	53	[illegible]
20	133	122	927	54	54	37	177	22	6	3	43	[illegible]
21	1,221	203	380	44	61	33	201	16	6	3	22	[illegible]
22	551	329	150	39	73	30	922	12	4	4	33	[illegible]
23	157	659	125	39	56	23	477	17	4	9	33	[illegible]
24	73	397	514	37	62	18	193	745	8	8	33	[illegible]
25	171	151	513	31	128	15	83	123	12	11	35	[illegible]
26	181	111	146	33	124	15	64	69	10	15	36	[illegible]
27	119	87	97	33	57	12	413	42	10	12	1,106	[illegible]
28	109	65	76	26	46	14	1,817	36	9	12	385	[illegible]
29	87	.....	61	24	35	14	1,292	21	5	21	148	[illegible]
30	97	.....	54	19	25	9	821	20	4	9	112	[illegible]
31	73	.....	47	.....	113	.....	116	25	.....	13	.....	[illegible]
Mean	160	287	194	142	411	157	233	65	11	7	163	[illegible]

Daily discharge, in second-feet, of Tohickon Creek at Point Pleasant, Pennsylvania, for 1898.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	133	110	82	251	151	65	4	7	7	5	113	364
2	131	99	67	131	103	53	5	13	7	3	60	271
3	90	98	70	98	79	44	4	8	7	5	46	201
4	49	95	79	82	71	35	3	10	8	5	37	1,230
5	49	90	79	80	73	31	5	28	5	5	31	2,982
6	36	89	68	83	109	31	6	96	5	7	31	683
7	55	87	64	111	404	27	3	44	5	5	27	466
8	87	80	61	103	2,940	14	3	39	14	3	31	139
9	87	73	53	73	1,711	15	3	39	14	4	28	87
10	87	73	49	65	398	15	3	326	9	6	1,508	79
11	87	65	43	68	229	16	2	574	9	6	1,697	71
12	144	450	43	73	337	15	4	167	7	6	357	65
13	239	634	47	63	557	15	3	74	7	9	146	53
14	216	327	43	60	246	20	3	40	7	4	107	49
15	452	253	39	338	1,201	16	3	35	8	6	89	51
16	696	198	39	289	847	16	5	32	6	8	60	46
17	251	138	39	154	789	11	5	25	7	8	60	35
18	107	132	39	89	299	9	6	17	6	17	378	23
19	87	154	39	71	145	9	5	134	5	23	3,451	27
20	375	4,160	39	68	125	9	7	88	4	23	896	472
21	748	1,737	57	64	117	10	17	64	4	24	279	531
22	367	860	376	61	91	9	20	39	6	79	136	237
23	2,890	427	469	53	69	5	13	27	8	70	811	2,134
24	680	231	304	569	297	5	7	23	13	52	518	536
25	226	159	283	544	573	5	11	21	15	66	283	251
26	279	115	175	323	767	5	11	22	5	224	210	131
27	320	97	131	238	495	7	11	14	7	495	185	107
28	439	97	242	525	266	7	11	6	10	171	194	89
29	370	.....	303	810	158	4	5	11	9	68	248	79
30	133	.....	1,078	322	119	4	8	11	7	94	341	73
31	259	.....	433	.....	89	.....	8	5	.....	169	.....	114
Mean	328	397	162	195	447	17	7	66	8	54	412	375

Daily discharge, in second-feet, of Tohickon Creek at Point Pleasant, Pennsylvania, for 1899.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	144	158	793	178	35	15	14	4	18	46	52	34
2	136	125	600	131	31	9	11	399	13	40	181	34
3	154	118	601	97	31	9	19	267	9	27	121	44
4	204	151	708	79	31	10	12	113	98	25	921	49
5	1,172	184	2,819	68	31	9	10	52	42	23	366	44
6	1,360	210	1,109	61	27	8	10	28	28	18	121	39
7	1,400	189	520	73	23	7	8	22	19	20	73	39
8	508	142	277	1,328	23	7	8	17	16	23	58	39
9	508	180	217	389	23	5	7	15	287	20	53	31
10	413	183	203	160	20	8	6	254	210	17	40	23
11	124	110	418	108	28	6	7	417	203	17	35	27
12	158	90	1,225	92	35	6	5	182	499	16	42	44
13	169	119	1,702	87	27	7	7	66	163	13	51	95
14	186	102	416	79	31	5	11	38	58	11	53	101
15	311	65	709	70	31	3	9	28	46	10	47	60
16	432	157	934	153	20	5	5	19	36	9	46	49
17	569	416	348	398	17	5	7	14	18	9	47	44
18	488	782	502	144	20	4	7	12	18	9	45	39
19	258	767	2,492	84	23	3	8	11	23	9	38	39
20	177	695	968	68	20	2	8	8	708	9	34	85
21	116	914	308	61	17	2	3	8	545	9	33	101
22	84	1,414	1,051	58	17	7	5	6	404	9	33	65
23	79	1,582	1,006	53	20	2	3	7	239	14	49	53
24	526	1,000	449	49	17	2	3	7	61	16	57	879
25	2,082	541	220	49	17	7	3	3	42	18	44	913
26	392	616	624	45	15	7	4	2	1,522	18	35	208
27	230	3,222	633	41	12	12	4	359	604	11	30	97
28	145	1,058	779	39	9	5	4	317	158	11	31	79
29	166	.....	1,307	39	12	17	6	62	78	10	36	65
30	169	.....	520	39	14	27	7	34	51	12	39	53
31	184	.....	239	.....	15	.....	4	27	.....	17	.....	49
Mean	419	546	797	144	22	7	7	90	271	17	94	113

NESHAMINY CREEK, PENNSYLVANIA, BELOW THE FORKS.

This station, which was established in 1884, is described in the Twentieth Annual Report, Part IV, pages 103 and 104. The same report contains also (pages 104 to 108) the figures for monthly flow for the years 1890 to 1898, inclusive, as well as diagrams of discharge for the same period. Water-Supply Paper No. 35 contains, on page 65, a table of daily discharge for 1899. The results by months and a diagram of the daily discharge for that year are given on page 86 of the Twenty-first Annual Report, Part IV. The following tables give the daily discharge of this stream from 1884 to 1900, inclusive.

Daily discharge, in second-feet, of Neshaminy Creek, Pennsylvania, below the forks, for 1884.

[Drainage area, 139.3 square miles.]

Day.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Day.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	.....	24	116	15	5	13	55	18	13	14	20	5	4	5	157
2	.....	19	101	11	5	14	44	19	9	18	21	5	6	5	175
3	.....	27	56	10	6	12	26	20	10	16	16	4	6	9	210
4	.....	25	57	10	6	8	20	21	8	13	18	4	5	10	363
5	.....	235	436	12	8	16	23	22	7	15	18	5	5	5	959
6	.....	905	462	8	8	18	235	23	5	16	17	5	5	3	940
7	.....	146	105	6	6	19	1,907	24	5	14	17	5	5	441	935
8	.....	76	69	6	5	15	192	25	6	15	17	5	4	110	811
9	.....	48	48	4	5	15	98	26	2,271	17	17	4	4	47	710
10	12	36	38	4	5	11	74	27	331	17	13	4	4	29	684
11	12	27	34	4	5	7	56	28	80	14	10	4	4	24	693
12	14	27	31	4	5	7	841	29	56	22	13	5	7	180	696
13	17	22	28	4	4	6	414	30	31	29	20	5	11	124	1,053
14	15	17	25	4	4	7	166	31	.....	36	18	.....	12	.....	1,161
15	12	19	22	3	3	8	2,354								
16	10	18	22	4	3	6	396	Mean	139	62	61	6	5	39	534
17	12	14	17	4	3	5	173								

Daily discharge, in second-feet, of Neshaminy Creek, Pennsylvania, below the forks, for 1885.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	666	139	74	334	46	19	7	3	9	1	29	117
2	215	128	284	222	52	22	4	4	6	1	722	97
3	125	142	343	137	55	14	4	723	4	1	207	93
4	120	146	492	1,761	47	15	4	2,377	3	8	91	88
5	130	156	1,053	450	44	15	3	94	5	6	55	76
6	1,886	150	948	206	38	9	4	39	7	5	44	109
7	823	137	456	154	63	10	41	17	4	6	38	122
8	342	138	208	168	474	13	28	18	3	6	190	128
9	252	1,714	137	181	222	13	10	17	4	5	746	109
10	204	4,484	80	120	141	11	7	16	5	4	179	216
11	151	1,067	84	117	114	11	7	16	2	7	106	157
12	1,377	631	78	121	78	10	4	15	4	2	82	76
13	355	260	65	115	66	8	3	11	5	42	71	667
14	190	156	114	93	57	6	4	11	5	67	70	1,816
15	220	131	136	87	44	6	4	7	4	28	57	457
16	653	3,454	169	88	44	9	3	9	4	16	44	213
17	1,026	2,423	160	80	44	10	5	6	4	11	43	167
18	233	209	84	80	37	7	3	7	3	8	38	151
19	268	133	85	67	34	5	3	6	4	5	35	144
20	249	96	69	50	32	9	4	6	3	7	28	111
21	173	89	60	47	28	8	4	6	4	63	27	97
22	190	85	52	46	28	6	5	9	2	108	23	104
23	242	82	48	42	31	4	3	7	4	54	325	98
24	236	76	46	42	31	4	3	4	3	31	670	97
25	227	81	42	50	26	6	2	4	2	16	543	84
26	182	87	78	55	29	7	2	6	3	11	378	99
27	197	54	138	55	26	5	2	8	2	11	267	134
28	255	33	126	54	20	5	2	7	1	11	164	116
29	231	.....	111	58	20	9	2	5	2	10	128	77
30	220	.....	161	54	21	9	2	5	1	44	134	63
31	192	.....	298	.....	22	.....	2	8	.....	54	.....	114
Mean	382	589	203	171	65	9	6	112	4	21	185	210

Daily discharge, in second-feet, of Neshaminy Creek, Pennsylvania, below the forks, for 1886.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	209	452	165	1,637	59	64	38	36	8	4	14	46
2	109	415	154	391	48	49	35	33	8	3	13	48
3	83	380	143	270	51	53	32	32	6	3	10	59
4	141	425	132	252	48	53	29	24	7	4	9	89
5	2,478	350	121	423	49	45	26	20	4	3	10	64
6	413	300	109	4,734	41	39	28	18	5	2	9	46
7	195	250	97	1,198	51	35	22	26	6	2	7	57
8	265	200	94	513	2,346	36	20	33	5	2	7	87
9	599	175	92	371	749	35	19	46	6	2	10	68
10	601	150	88	303	227	45	182	31	4	2	11	74
11	574	5,767	78	231	168	49	149	24	4	4	11	85
12	417	5,132	77	207	140	37	48	18	4	2	10	64
13	346	4,195	89	296	218	29	32	19	3	2	85	105
14	322	830	77	319	566	194	27	16	4	2	74	342
15	307	508	84	207	232	291	44	13	5	2	27	345
16	300	528	75	168	615	100	294	12	10	2	27	393
17	285	217	70	157	242	95	297	16	7	2	16	279
18	272	173	66	139	159	108	100	11	7	2	26	649
19	285	204	79	133	164	67	136	15	4	3	37	2,228
20	287	199	93	120	167	39	90	11	4	2	26	671
21	372	202	902	108	155	34	58	9	6	3	17	278
22	660	146	477	99	131	31	75	6	5	2	16	190
23	580	127	216	93	173	732	62	6	4	3	111	133
24	375	122	141	90	134	315	48	6	4	3	203	203
25	330	694	132	85	113	149	31	7	3	3	160	662
26	321	1,106	122	93	108	276	60	8	6	4	770	212
27	390	189	128	86	89	111	598	6	3	11	124	175
28	1,744	177	439	81	96	73	114	6	4	28	64	145
29	3,735	.....	456	76	85	63	73	4	2	38	52	228
30	1,269	.....	1,389	65	67	45	42	9	2	25	50	178
31	615	.....	1,870	.....	59	.....	41	9	.....	15	.....	229
Mean	609	843	266	431	244	110	92	17	5	6	67	272

Daily discharge, in second-feet, of Neshaminy Creek, Pennsylvania, below the forks, for 1887.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	323	239	296	204	115	208	42	144	21	47	37	40
2	580	156	262	202	105	137	41	149	24	33	34	39
3	434	220	335	228	96	104	34	218	22	38	28	26
4	344	208	195	222	87	101	30	139	17	31	27	29
5	254	204	162	177	83	53	132	88	23	23	22	23
6	221	165	186	147	76	39	1,570	73	19	22	25	55
7	193	612	324	124	160	107	223	73	20	18	19	48
8	181	742	719	121	310	158	135	65	18	18	23	40
9	138	625	545	111	244	84	150	50	23	20	18	29
10	181	337	828	107	148	54	942	44	18	21	24	176
11	181	767	405	107	118	44	146	108	14	21	52	1,648
12	159	552	221	92	91	35	89	236	492	19	47	400
13	153	189	213	81	77	31	66	67	229	14	28	199
14	740	223	196	77	69	26	53	44	83	19	21	153
15	940	597	160	78	56	23	47	41	54	17	33	610
16	420	504	154	80	56	28	40	41	43	14	28	607
17	230	370	139	69	55	24	56	36	28	13	29	210
18	300	1,344	129	210	48	35	58	92	26	18	23	277
19	210	1,708	120	359	47	50	39	71	27	13	26	308
20	160	430	110	118	38	34	40	40	23	18	31	249
21	130	321	109	134	34	29	35	30	26	398	51	411
22	220	406	413	108	33	411	38	37	24	139	44	447
23	590	376	551	294	32	3,159	462	64	22	63	37	289
24	2,891	630	437	663	41	412	1,047	379	20	43	31	248
25	628	314	691	203	53	179	484	288	21	36	29	169
26	490	294	432	275	86	119	177	84	20	34	27	175
27	532	1,210	255	205	44	102	282	45	20	34	26	145
28	400	454	2,185	153	39	81	134	42	26	29	29	1,597
29	1,661	.....	521	191	25	62	147	39	41	31	61	913
30	900	.....	246	154	33	51	121	32	74	27	38	364
31	408	.....	235	.....	32	.....	141	31	.....	32	.....	435
Mean	490	507	380	176	82	199	226	93	51	42	32	334

Daily discharge, in second-feet, of Neshaminy Creek, Pennsylvania, below the forks, for 1888.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	4,890	233	160	376	77	65	19	10	12	68	131	178
2	2,786	235	145	306	72	42	13	11	13	75	124	182
3	505	230	139	262	72	39	15	11	15	65	114	140
4	388	285	114	211	72	36	13	9	15	50	106	135
5	323	1,553	95	295	63	31	13	7	12	47	97	131
6	267	2,111	162	2,227	51	31	15	7	9	191	93	139
7	310	798	77	428	51	29	13	7	9	212	87	136
8	636	389	101	354	54	22	10	17	224	130	84	129
9	404	308	95	206	48	25	14	21	376	74	91	118
10	261	310	88	631	48	24	22	14	134	57	171	142
11	251	313	602	1,394	78	21	21	11	129	52	347	145
12	432	283	1,571	341	129	23	14	78	474	53	157	118
13	682	230	338	271	108	17	11	206	280	68	105	80
14	493	220	375	242	75	15	10	154	174	69	100	173
15	356	220	432	197	61	17	9	22	88	63	1,821	248
16	287	235	564	781	58	19	11	20	84	53	588	131
17	342	267	743	557	57	14	12	15	1,052	69	301	3,750
18	455	362	565	200	48	10	11	9	3,099	63	209	1,822
19	480	587	367	171	52	16	10	7	401	63	877	401
20	477	2,112	1,001	160	61	20	11	6	227	68	560	311
21	288	2,675	3,126	154	56	20	11	29	1,710	69	258	220
22	199	579	1,455	125	40	14	8	1,500	319	54	193	253
23	206	411	350	119	36	65	10	153	185	60	166	365
24	251	265	264	119	40	57	11	55	151	101	153	354
25	172	3,619	248	97	40	14	9	36	132	132	150	302
26	151	938	449	88	45	15	9	15	135	82	245	209
27	186	368	1,387	80	43	11	65	19	125	406	671	351
28	175	228	783	80	42	8	125	20	92	688	345	553
29	195	191	1,485	72	95	15	24	15	72	459	218	329
30	226	.....	578	68	130	24	19	14	64	209	196	157
31	232	.....	489	.....	91	.....	19	13	.....	151	.....	135
Mean	558	709	592	350	64	25	18	80	327	129	291	240

Daily discharge, in second-feet, of Neshaminy Creek, Pennsylvania, below the forks, for 1889.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	139	183	101	87	214	312	94	1,760	53	185	244	324
2	140	140	114	558	182	298	346	1,083	58	179	216	323
3	131	154	484	250	154	158	649	1,047	51	155	616	293
4	116	159	3,197	198	138	120	905	761	53	133	526	249
5	197	144	1,364	156	122	122	598	705	44	126	254	236
6	940	120	503	139	108	116	247	480	38	117	212	197
7	490	214	296	120	105	95	168	595	41	123	186	172
8	260	328	215	111	89	88	135	296	50	106	180	156
9	552	188	169	111	80	96	113	228	41	94	2,879	333
10	776	116	142	96	80	97	94	321	44	94	1,044	160
11	275	102	133	92	72	93	124	239	57	91	461	176
12	200	110	131	88	71	143	158	174	108	87	364	162
13	166	124	131	92	77	120	490	152	777	267	1,022	233
14	157	133	137	84	108	141	332	746	381	208	1,179	253
15	145	118	138	72	134	89	3,386	2,046	835	401	276	216
16	144	396	130	65	86	172	425	188	463	213	296	196
17	800	738	115	61	59	118	237	226	2,560	137	253	192
18	449	1,116	115	68	50	301	173	186	2,947	113	253	192
19	227	294	115	60	41	108	180	163	618	98	2,965	192
20	148	177	116	51	822	61	3,950	152	429	92	868	192
21	970	125	263	59	781	57	401	136	376	116	547	192
22	497	92	937	68	301	54	263	133	281	138	888	192
23	286	88	475	62	180	43	208	132	216	133	409	192
24	255	97	253	61	127	40	169	149	198	172	324	192
25	387	110	204	48	100	40	148	130	990	137	668	192
26	394	99	169	689	215	562	132	110	564	102	458	192
27	1,130	89	146	2,258	738	400	201	105	384	3,749	1,171	192
28	872	87	143	1,121	538	148	231	89	242	967	3,538	192
29	361	.....	141	501	408	92	174	72	201	388	415	192
30	213	.....	122	284	330	80	528	65	184	274	407	192
31	214	.....	89	.....	141	.....	5,531	49	.....	232	.....	192
Mean	356	208	348	257	214	145	658	410	443	304	783	192

Daily discharge, in second-feet, of Neshaminy Creek, Pennsylvania, below the forks, for 1890.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	107	135	722	346	115	95	30	38	46	28	145	52
2	110	114	347	351	87	80	33	41	31	32	135	72
3	108	157	306	250	68	53	135	27	27	391	130	91
4	101	163	197	610	64	53	566	20	25	188	123	137
5	108	192	178	626	114	48	107	31	26	88	102	144
6	130	147	282	272	138	65	56	21	26	67	90	102
7	128	111	229	228	191	65	44	27	20	135	90	116
8	113	2,848	141	270	132	43	42	19	18	129	89	192
9	97	974	132	666	175	35	29	28	21	133	85	190
10	90	293	149	639	96	35	27	17	18	78	79	103
11	92	250	581	280	75	112	19	16	19	83	116	81
12	80	228	560	219	76	1,150	17	16	26	75	140	85
13	84	212	282	180	78	667	15	13	29	64	114	165
14	84	526	1,203	159	70	272	17	10	29	77	95	106
15	1,591	601	1,309	149	142	279	30	10	55	74	85	74
16	800	237	544	139	223	225	22	14	180	64	75	88
17	281	192	323	119	87	75	19	27	147	564	118	276
18	187	189	290	105	62	70	60	27	61	188	231	848
19	150	339	261	96	49	60	33	27	50	130	120	193
20	281	1,101	315	88	165	51	19	26	38	136	84	355
21	298	331	1,939	86	264	65	19	222	31	136	85	182
22	144	197	2,990	85	94	76	21	670	29	146	81	106
23	170	266	1,512	71	63	56	20	118	20	585	73	115
24	159	213	500	74	52	41	17	50	20	2,400	68	105
25	155	266	404	97	47	48	21	38	19	783	72	120
26	110	278	473	101	192	43	415	37	27	341	68	165
27	119	210	320	96	1,986	39	180	163	40	240	66	201
28	119	357	1,419	104	309	29	59	48	33	198	52	189
29	101	.....	635	87	168	27	113	56	23	203	70	173
30	162	.....	367	61	126	27	54	54	29	204	52	157
31	182	.....	313	.....	102	.....	34	58	.....	167	.....	130
Mean	201	397	620	222	181	133	73	63	39	262	98	165

Daily discharge, in second-feet, of Neshaminy Creek, Pennsylvania, below the forks, for 1891.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	128	1,585	370	379	51	34	16	77	164	27	41	91
2	1,953	773	242	252	41	35	17	80	128	22	37	74
3	844	739	212	931	57	27	20	89	101	17	39	50
4	275	588	289	362	83	26	24	53	85	17	29	347
5	371	263	269	247	57	30	20	39	555	18	35	964
6	605	200	196	206	56	27	15	36	1,939	22	43	199
7	527	334	202	184	57	28	16	30	341	26	37	920
8	376	623	266	155	41	38	22	21	203	26	39	443
9	308	786	994	152	36	37	26	18	154	26	39	205
10	253	981	1,291	137	40	28	22	33	129	26	35	167
11	1,377	423	443	226	34	26	20	33	106	21	54	142
12	1,594	306	689	402	38	22	22	20	93	19	75	167
13	354	261	1,662	226	39	20	17	20	84	23	75	99
14	185	225	538	164	31	18	19	17	77	17	57	99
15	147	193	275	140	30	23	21	40	63	12	39	100
16	140	1,410	303	131	51	22	19	56	54	19	39	105
17	283	1,156	242	136	47	25	17	38	51	19	48	97
18	805	640	213	134	28	126	16	38	45	19	98	94
19	279	350	201	114	31	58	127	299	39	17	87	93
20	223	360	266	104	33	41	91	86	34	328	60	61
21	191	906	3,446	93	26	36	40	47	31	194	48	57
22	3,257	1,135	789	93	31	42	29	27	27	155	42	76
23	768	415	1,053	93	36	38	26	43	27	340	91	1,495
24	356	332	512	86	31	29	26	3,276	27	121	261	1,630
25	930	334	358	77	26	73	24	1,264	27	63	123	1,040
26	642	569	288	55	26	20	19	278	31	48	94	5,218
27	506	553	248	55	30	19	14	182	27	101	109	650
28	430	368	239	66	30	15	19	335	22	109	239	261
29	1,084	.....	222	62	37	16	150	208	27	58	105	299
30	1,679	.....	196	67	34	16	214	267	31	52	81	1,070
31	935	.....	182	.....	31	.....	122	248	.....	52	.....	674
Mean	703	600	525	182	39	31	40	235	157	65	73	366

Daily discharge, in second-feet, of Neshaminy Creek, Pennsylvania, below the forks, for 1892.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	999	191	2,582	179	79	118	39	69	15	5	6	149
2	1,870	179	554	167	107	102	82	51	14	6	6	106
3	743	174	329	179	133	103	297	51	12	5	6	102
4	233	159	259	174	114	99	519	35	11	4	7	80
5	242	149	269	154	86	92	144	20	10	4	7	77
6	194	144	221	144	73	111	73	23	13	4	5	68
7	200	135	207	133	73	107	53	20	13	4	5	59
8	192	129	1,359	121	55	79	42	18	8	5	6	63
9	180	131	942	119	50	93	34	17	6	6	8	101
10	178	122	389	110	55	140	31	17	6	5	32	93
11	144	120	309	99	55	130	31	21	7	4	99	38
12	146	115	257	96	50	80	31	60	7	5	43	51
13	3,584	125	214	92	50	58	31	52	8	5	19	60
14	2,364	121	155	102	59	53	31	25	96	6	19	417
15	767	93	159	186	437	48	48	17	17	6	734	559
16	408	81	149	193	215	39	31	19	15	5	2,346	226
17	363	81	130	108	127	34	31	15	15	5	234	163
18	444	81	125	97	84	36	22	18	11	8	605	112
19	2,172	77	135	99	144	36	31	17	10	8	643	85
20	731	82	144	87	459	31	24	15	10	5	184	130
21	382	93	135	93	341	32	16	13	10	4	114	220
22	322	115	114	152	419	32	14	10	10	4	86	97
23	280	124	117	185	492	29	14	11	9	5	89	122
24	250	108	386	161	297	26	15	12	9	5	56	214
25	257	103	502	109	177	72	16	16	13	6	45	122
26	280	159	409	94	165	54	16	21	19	7	39	241
27	332	184	900	90	756	75	15	22	10	5	54	119
28	331	125	1,026	88	376	109	15	19	7	5	241	101
29	286	117	440	88	187	69	19	19	4	5	611	106
30	273	.....	251	88	154	37	191	16	3	4	243	115
31	233	.....	209	.....	142	.....	101	13	.....	5	.....	139
Mean	627	125	431	126	194	71	66	24	13	5	220	127

Daily discharge, in second-feet, of Neshaminy Creek, Pennsylvania, below the forks, for 1893.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	1,357	385	463	133	362	57	23	14	59	26	61	173
2	1,678	568	599	116	318	57	23	11	54	21	59	154
3	227	741	481	102	646	57	25	10	48	23	59	217
4	150	692	281	109	3,154	46	21	7	39	24	1,281	1,938
5	125	540	710	98	571	54	18	7	35	21	1,759	316
6	125	484	199	85	894	73	18	7	28	20	352	274
7	100	1,590	169	98	420	154	17	9	29	22	221	211
8	90	714	326	153	512	75	20	11	34	21	188	253
9	130	307	2,337	140	652	51	21	9	31	30	241	186
10	135	2,727	2,394	177	404	45	20	7	27	27	179	305
11	120	869	943	380	188	39	23	7	27	24	139	325
12	150	398	2,436	198	176	36	22	6	31	24	124	146
13	140	861	920	153	159	39	19	6	31	23	115	144
14	140	1,299	683	544	149	41	17	5	31	70	115	258
15	130	1,136	895	2,448	144	39	15	6	53	77	303	236
16	135	858	358	467	273	36	15	8	749	34	234	1,323
17	140	495	302	290	617	36	18	9	258	28	121	299
18	176	625	273	229	223	35	19	12	107	25	120	330
19	168	371	251	198	161	33	15	11	78	21	109	325
20	175	395	236	742	135	27	15	174	56	20	94	147
21	165	484	244	1,012	117	24	14	254	48	21	101	170
22	154	404	244	831	108	29	12	63	41	18	330	169
23	154	385	232	293	105	284	9	29	36	199	213	146
24	154	309	272	235	105	90	9	1,692	32	283	135	144
25	149	258	235	213	89	42	11	197	29	119	99	152
26	154	179	232	221	72	39	11	80	39	64	91	154
27	147	194	184	418	72	45	10	43	33	40	112	150
28	136	242	156	362	72	35	8	31	29	481	1,500	142
29	164	.....	139	232	77	35	14	1,078	33	206	996	119
30	215	.....	133	205	77	29	9	214	30	104	210	133
31	321	.....	133	.....	63	.....	8	90	.....	88	.....	124
Mean	242	661	565	363	358	56	16	132	72	70	322	318

Daily discharge, in second-feet, of Neshaminy Creek, Pennsylvania, below the forks, for 1894.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	107	115	198	128	97	368	68	32	7	40	428	115
2	109	110	227	110	86	285	43	19	8	47	196	268
3	100	90	882	97	98	247	37	400	8	41	1,371	315
4	89	58	936	111	95	233	34	145	6	45	640	161
5	98	60	776	189	80	231	36	48	6	71	860	133
6	130	58	777	142	68	280	165	31	6	65	866	124
7	136	57	698	112	76	315	177	56	5	43	245	110
8	116	60	416	115	80	235	75	61	857	34	342	229
9	105	1,063	313	114	68	194	51	28	1,247	41	464	616
10	97	757	291	113	57	161	40	14	102	1,715	313	257
11	88	517	277	146	43	144	34	16	108	380	258	629
12	88	267	243	363	39	130	36	15	78	173	198	1,824
13	88	164	380	1,521	39	127	34	67	36	164	179	853
14	76	164	355	833	42	127	29	82	28	250	175	241
15	73	130	171	490	45	116	26	24	28	172	165	300
16	88	250	178	303	39	97	101	23	23	110	149	257
17	92	114	171	239	36	92	212	20	19	106	170	228
18	82	1,630	144	198	36	94	60	15	1,001	90	227	194
19	78	1,990	144	174	36	87	31	11	3,197	78	176	172
20	81	1,185	141	166	685	77	31	16	797	72	145	159
21	79	603	136	188	9,012	72	31	22	245	63	150	147
22	83	399	179	418	4,036	69	23	22	148	59	170	149
23	88	366	571	291	823	60	21	11	111	59	154	145
24	88	287	279	186	3,779	51	26	15	91	60	152	125
25	96	204	194	159	1,523	51	29	13	81	200	143	135
26	80	105	182	141	532	82	27	14	81	233	125	135
27	75	147	159	122	277	89	22	14	62	125	120	135
28	88	200	131	114	2,893	60	22	10	46	93	112	135
29	67	.....	149	109	2,036	45	19	8	48	82	108	130
30	108	.....	164	105	553	65	19	7	40	79	96	130
31	188	.....	152	.....	443	.....	44	7	.....	790	.....	111
Mean	95	37	323	250	895	143	52	41	284	180	296	279

Daily discharge, in second-feet, of Neshaminy Creek, Pennsylvania, below the forks, for 1895.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	155	156	1,217	160	192	46	528	27	8	6	27	15
2	98	98	1,019	249	145	42	148	20	8	6	31	27
3	117	106	584	367	118	82	74	18	9	4	30	76
4	98	147	533	231	98	95	59	1,225	10	5	15	50
5	98	201	475	163	87	95	664	526	11	5	17	45
6	156	189	265	142	88	157	616	94	8	5	16	51
7	357	176	321	124	88	126	202	54	7	5	17	40
8	891	176	1,531	2,542	82	56	115	43	7	6	12	25
9	818	176	694	3,234	74	41	92	34	6	7	11	50
10	579	188	594	584	72	38	73	29	6	7	9	56
11	2,289	188	605	359	63	41	49	20	6	7	8	27
12	830	176	273	280	56	48	44	71	6	8	7	16
13	550	176	723	661	65	134	48	70	6	16	9	8
14	349	188	1,632	755	76	145	40	40	6	29	8	6
15	295	178	396	338	78	62	35	29	5	34	8	6
16	295	177	292	253	71	44	30	22	6	27	9	6
17	295	175	240	204	59	36	48	22	7	22	7	6
18	150	182	376	182	53	34	48	17	8	19	7	7
19	150	201	625	168	47	31	39	13	7	15	13	10
20	164	242	526	152	41	28	26	19	7	13	9	10
21	188	323	287	136	65	25	19	20	7	8	10	9
22	191	385	363	130	88	26	32	14	6	7	9	89
23	166	385	253	130	71	31	39	13	5	6	7	81
24	150	258	292	118	48	29	39	10	5	5	8	43
25	120	242	445	107	41	25	36	9	5	5	7	27
26	2,053	287	450	101	39	49	28	9	5	5	11	23
27	479	368	247	127	165	89	24	13	5	5	34	32
28	203	903	213	261	225	202	21	13	4	5	28	37
29	178	.....	190	159	105	106	18	11	6	6	18	32
30	178	.....	160	187	63	78	25	10	6	6	15	24
31	178	.....	149	.....	48	.....	31	10	.....	9	.....	573
Mean	412	237	515	417	84	68	106	81	7	10	14	49

Daily discharge, in second-feet, of Neshaminy Creek, Pennsylvania, below the forks, for 1896.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	186	39	960	286	81	24	27	71	11	155	28	280
2	83	45	463	286	65	23	23	48	8	60	30	198
3	69	51	99	260	51	22	24	37	8	38	33	115
4	72	51	57	217	63	22	24	385	10	31	36	115
5	55	63	57	172	59	17	19	30	75	29	1,305	100
6	32	8,707	101	154	39	17	17	24	673	33	579	90
7	48	2,535	127	164	44	20	22	25	214	29	220	90
8	36	600	111	154	41	17	37	30	65	24	106	120
9	23	635	88	135	30	18	57	31	32	26	289	307
10	19	624	85	129	23	66	36	22	23	24	119	282
11	19	383	82	129	25	46	71	17	20	19	115	154
12	15	228	102	125	38	31	29	25	15	44	154	120
13	14	187	91	125	42	24	22	23	14	1,480	169	125
14	14	282	79	120	34	196	20	28	17	345	135	100
15	15	379	92	110	31	236	17	74	14	206	102	95
16	15	274	81	93	31	73	17	36	13	129	88	90
17	15	121	77	82	31	52	17	22	503	100	88	95
18	19	119	118	93	31	138	16	17	765	74	85	95
19	19	110	5,408	92	31	72	13	17	290	62	80	80
20	23	109	1,994	81	31	35	13	17	169	51	73	75
21	31	88	715	135	31	29	13	14	57	53	66	75
22	27	90	414	147	31	26	1,373	11	40	51	69	60
23	32	85	309	97	31	29	499	10	33	43	72	55
24	565	75	250	88	31	31	166	14	26	61	72	54
25	377	84	211	97	31	36	92	14	26	63	72	52
26	142	55	349	97	31	54	68	10	24	59	68	50
27	59	66	403	88	31	50	77	10	19	53	64	50
28	53	70	257	85	110	32	97	14	17	37	143	50
29	59	1,542	2,061	81	135	43	180	15	21	33	869	45
30	43	.....	760	81	61	43	561	12	308	38	285	40
31	36	.....	389	.....	36	.....	155	13	.....	40	.....	30
Mean	71	611	528	133	46	50	126	25	120	112	190	106

Daily discharge, in second-feet, of Neshaminy Creek, Pennsylvania, below the forks, for 1897.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	88	41	88	93	76	445	45	221	43	17	80	105
2	88	41	115	77	1,610	115	42	184	41	13	1,354	94
3	121	41	169	71	439	84	42	115	41	13	287	91
4	209	41	200	65	289	124	41	105	37	13	108	118
5	351	41	190	131	170	273	37	105	32	17	81	1,305
6	233	1,094	360	166	135	130	32	97	31	17	71	388
7	125	2,973	208	125	111	84	29	85	31	15	42	204
8	125	840	144	99	92	103	29	77	31	15	36	155
9	107	334	144	1,750	80	4,677	31	80	31	12	61	144
10	80	166	160	606	75	774	31	83	27	10	74	138
11	72	135	160	272	97	224	27	86	28	13	60	138
12	72	115	220	205	96	216	27	83	30	15	55	236
13	57	110	220	166	2,215	228	36	63	28	15	49	214
14	41	94	171	144	755	202	75	54	29	15	39	2,436
15	48	72	215	208	800	159	61	64	27	15	37	2,606
16	48	63	149	218	235	144	36	289	24	14	40	492
17	41	54	129	164	190	137	31	160	23	11	41	318
18	41	48	135	145	150	181	31	67	22	13	41	245
19	41	41	179	125	140	124	140	54	19	15	41	197
20	93	48	700	107	126	117	356	63	19	15	40	175
21	1,356	131	328	88	164	112	367	66	21	15	40	175
22	369	529	209	88	165	97	3,083	64	23	14	41	173
23	139	1,826	170	88	117	76	1,256	53	24	14	39	162
24	97	301	469	88	127	73	690	1,075	24	14	39	154
25	73	105	327	80	711	85	391	209	27	14	38	154
26	48	89	189	84	185	74	181	119	27	59	36	154
27	48	63	154	84	110	53	491	93	28	85	36	154
28	36	63	139	72	84	59	2,119	73	21	43	1,053	171
29	31	.....	129	66	74	56	900	60	21	27	238	166
30	31	.....	125	66	77	48	298	50	21	23	142	154
31	36	.....	115	.....	628	.....	214	49	21	19	124	539
									.....	16	.....	
Mean	156	339	210	191	331	307	358	130	27	20	147	305

Daily discharge, in second-feet, of Neshaminy Creek, Pennsylvania, below the forks, for 1898.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	377	164	164	249	193	115	29	32	24	12	65	436
2	340	164	160	150	162	105	29	251	17	8	39	300
3	340	164	160	176	147	105	21	65	14	6	31	288
4	233	164	164	159	138	97	15	41	14	7	24	1,777
5	135	164	154	164	129	117	15	105	13	7	19	3,059
6	125	164	144	164	140	110	23	119	14	7	17	511
7	105	164	144	164	362	69	25	36	14	6	25	304
8	101	164	139	199	3,153	62	22	22	16	8	27	218
9	92	189	131	131	846	62	22	22	20	7	24	147
10	88	213	127	122	378	55	26	32	17	7	933	144
11	97	294	122	120	303	45	15	397	12	9	852	135
12	188	566	115	133	349	43	16	99	13	10	229	125
13	257	530	109	123	483	48	17	57	11	14	165	115
14	215	258	106	115	254	51	17	36	9	13	125	105
15	633	228	97	195	1,473	45	16	32	8	10	125	105
16	553	313	88	136	1,060	45	22	27	10	7	85	105
17	264	105	88	125	983	48	19	22	14	8	161	105
18	166	130	90	105	386	42	14	228	11	8	373	93
19	139	387	87	105	329	31	17	1,615	7	14	3,529	85
20	493	5,076	77	105	349	34	14	278	6	13	552	1,006
21	654	1,205	108	93	439	48	58	113	6	24	268	293
22	398	691	196	81	258	31	66	65	7	49	202	480
23	2,985	445	349	77	213	28	36	45	9	49	828	1,670
24	476	330	273	294	258	34	22	36	11	27	484	408
25	313	273	164	400	280	34	18	31	13	24	355	203
26	554	221	166	254	250	36	20	31	12	59	269	149
27	462	200	175	189	242	31	22	31	14	176	242	129
28	267	182	246	645	209	34	22	27	14	87	445	115
29	215	.....	391	782	170	34	19	22	11	40	322	110
30	176	.....	1,232	574	157	29	17	26	13	28	404	90
31	164	.....	420	.....	138	.....	14	29	.....	81	.....	161
Mean	374	469	199	211	459	55	23	128	12	27	376	418

Daily discharge, in second-feet, of Neshaminy Creek, Pennsylvania, below the forks, for 1899.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	527	213	611	349	67	31	15	27	15	36	144	54
2	453	213	569	280	73	31	13	248	15	31	154	51
3	97	213	540	243	67	27	9	1,262	28	31	162	48
4	71	213	686	221	61	19	10	104	41	27	1,257	51
5	405	173	3,687	201	53	17	14	43	32	22	252	54
6	2,105	119	913	176	50	17	13	23	19	38	145	48
7	1,298	97	593	255	47	13	10	15	15	59	107	41
8	306	97	377	1,047	45	13	13	15	15	53	85	41
9	277	105	388	362	46	15	14	14	13	36	81	36
10	213	97	343	245	38	19	14	488	43	27	81	36
11	213	80	510	204	64	19	13	2,577	38	22	81	36
12	213	72	959	186	89	17	10	151	80	19	85	36
13	213	72	1,318	176	73	17	10	71	68	19	85	65
14	228	72	407	166	53	15	10	48	23	22	77	80
15	203	72	1,172	159	48	15	8	36	15	19	72	60
16	189	89	813	193	48	15	8	27	15	19	68	45
17	457	319	242	301	43	15	9	22	15	22	68	36
18	252	553	722	177	54	15	9	19	13	123	68	31
19	145	574	2,490	149	68	15	8	19	41	113	56	36
20	115	974	742	139	57	15	6	17	430	57	51	57
21	105	1,373	382	129	36	15	6	14	166	41	48	72
22	105	1,373	1,765	115	36	15	6	19	138	32	41	57
23	100	1,402	976	105	45	15	6	19	65	27	77	41
24	1,117	849	405	105	51	13	6	19	41	27	147	792
25	1,927	446	414	105	48	11	6	19	41	22	90	401
26	333	809	490	105	36	13	11	13	578	22	74	125
27	242	3,950	417	101	31	13	23	11	204	22	72	89
28	213	815	1,398	92	31	13	27	13	89	17	63	72
29	213	.....	2,417	88	27	14	22	13	63	24	63	72
30	213	.....	588	73	22	15	325	14	48	17	63	63
31	213	.....	443	.....	27	.....	68	15	.....	24	.....	48
Mean	411	551	895	208	49	16	23	174	80	34	130	89

Daily discharge, in second-feet, of Neshaminy Creek, Pennsylvania, below the forks, for 1900.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	60	173	2,675	105	64	47	31	36	17	15	11	39
2	63	194	1,434	105	52	41	27	31	11	11	11	34
3	48	182	725	110	40	41	22	27	10	8	13	31
4	57	188	431	110	44	41	22	22	10	14	15	711
5	80	2,302	209	105	40	37	52	19	10	10	15	635
6	88	533	205	105	33	31	59	15	9	10	15	153
7	88	521	224	102	36	26	42	15	8	9	15	123
8	88	1,330	228	97	40	27	51	15	8	8	13	102
9	88	1,244	213	96	41	39	54	15	8	11	10	59
10	88	339	201	84	44	45	43	15	8	15	13	54
11	97	322	176	76	45	41	27	14	8	13	13	54
12	219	379	154	84	43	43	27	14	6	10	13	54
13	2,095	2,235	135	87	41	38	31	78	5	10	11	48
14	310	566	115	84	35	31	29	23	5	118	8	41
15	374	358	105	78	31	33	24	16	7	52	9	41
16	251	340	105	76	30	33	22	15	58	31	9	41
17	250	191	147	82	26	36	20	15	27	27	8	26
18	213	142	176	101	107	36	18	15	16	19	8	26
19	250	142	154	470	3,990	31	18	13	10	15	8	22
20	295	173	1,972	860	2,449	31	14	10	10	15	9	22
21	2,588	139	457	583	277	31	10	9	10	15	9	22
22	841	3,163	214	230	134	24	10	14	8	15	8	22
23	288	1,865	175	147	139	26	21	10	6	15	9	22
24	183	930	139	116	216	24	27	13	8	14	9	22
25	160	463	173	115	137	22	37	15	10	13	9	19
26	153	338	115	99	156	19	250	11	8	11	840	15
27	219	355	101	85	115	1,221	94	14	6	10	223	19
28	237	375	189	68	82	860	43	13	6	10	71	22
29	147	.....	146	64	61	104	23	85	6	13	48	22
30	105	.....	116	64	54	48	232	88	11	13	41	22
31	119	.....	105	.....	53	.....	73	22	.....	10	.....	20
Mean	327	696	378	153	279	102	47	23	11	18	50	91

PERKIOMEN CREEK AT FREDERICK, PENNSYLVANIA.

Measurements of this creek were begun on August 20, 1884. The station is described in the Twentieth Annual Report, Part IV, pages 89 and 90, followed by tables of monthly flow for the years 1890 to 1899, inclusive. Water-Supply Paper No. 35 contains tables of the daily discharge for the entire period from 1884 to 1899, inclusive. Diagrams of daily discharge, constructed from these tables, were published in the Twenty-first Annual Report, Part IV, pages 79 and 80, and figures for the monthly flow for 1899 on page 78 of that report. The following records of daily discharge for 1900 are furnished by Mr. John E. Codman, hydrographer of the water department of Philadelphia.

Daily discharge, in second-feet, of Perkiomen Creek at Frederick, Pennsylvania, for 1900.

Day.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	52	168	360	148	84	52	29	84	41	36	41	58
2	53	178	1,648	158	80	55	24	64	31	36	36	55
3	43	178	1,212	148	102	55	34	54	29	36	31	44
4	35	186	371	138	110	46	62	45	36	31	31	270
5	32	3,606	190	133	84	52	71	51	31	26	36	565
6	30	772	342	123	60	52	71	47	29	58	36	197
7	38	284	368	115	56	46	52	36	29	47	41	107
8	36	1,505	219	98	63	49	38	36	26	36	41	75
9	43	1,370	203	93	67	44	31	36	31	33	38	67
10	41	198	188	93	75	23	36	34	31	34	43	58
11	64	198	193	88	63	43	36	24	22	37	32	58
12	2,071	198	208	135	58	63	64	36	14	31	46	52
13	460	3,112	208	163	52	54	52	41	23	26	67	43
14	123	772	208	183	56	52	38	31	36	39	42	43
15	106	284	219	101	63	55	26	26	47	63	31	43
16	135	203	255	98	58	54	24	26	58	67	31	38
17	138	198	260	145	58	51	22	31	44	52	29	36
18	101	188	260	362	58	46	24	122	33	46	24	36
19	126	168	267	1,212	585	46	41	198	31	46	20	36
20	2,342	158	699	315	693	49	41	93	31	46	24	41
21	1,147	146	415	195	246	49	24	63	33	41	31	41
22	358	2,501	273	178	129	46	18	54	31	36	36	36
23	249	1,984	255	173	98	46	18	41	33	36	31	44
24	196	615	267	158	88	37	255	31	38	31	31	55
25	185	442	193	138	102	34	690	31	36	26	41	63
26	203	375	178	115	102	38	1,505	68	33	26	150	63
27	188	389	168	93	80	41	245	80	36	26	222	47
28	178	360	158	84	67	46	116	58	36	26	123	47
29	168	.....	133	84	71	41	71	52	36	26	58	47
30	158	.....	118	84	67	36	52	46	36	31	58	36
31	158	.....	128	.....	58	.....	135	43	.....	41	.....	207
Mean	299	741	321	178	117	47	127	54	33	38	50	84

[Continued in Water-Supply and Irrigation Paper No. 48.]

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